SURVEY DATA

ESTIMATE AND DESIGN DATA

ESTIMATE AND DESIGN DATA (*Continued*)

CONSTRUCTION DATA

HANDBOOK FOR HIGHWAY ENGINEERS

VOLUME I

PRINCIPLES AND PRACTICE

OTHER BOOKS

BY

W. G. HARGER

Rural Highway Pavements
613 pages, 6 × 9, 213 Illustrations.

Location, Grading and Drainage of Highways
294 pages, 6 × 9, 134 Illustrations.

HANDBOOK

FOR

HIGHWAY ENGINEERS

CONTAINING INFORMATION ORDINARILY USED
IN THE DESIGN AND CONSTRUCTION
OF RURAL HIGHWAYS

VOLUME I

PRINCIPLES AND PRACTICE

BY

WILSON G. HARGER, C. E.

AND

EDMUND A. BONNEY

FORMERLY SUPERVISING ENG., N. Y. STATE DEPT. OF HIGHWAYS

FOURTH EDITION
ENTIRELY REVISED AND ENLARGED
TWELFTH IMPRESSION

McGRAW-HILL BOOK COMPANY, INC.
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PREFACE TO FOURTH EDITION

This edition contains the selected data which 14 years use of the older editions indicates as desirable to meet the requirements of the everyday work of Highway Engineers. It includes information ordinarily needed for General Planning and for the detail work of Surveys, Design and Construction.

A Revision is necessary at this time on account of the rapid strides made in Rural Road Practice to meet the increased demands of modern motor traffic. The text has been entirely rewritten. Construction and Reconstruction methods and costs have been brought up to date; General Economics and Railroad Grade Crossing protection data have been added at the request of users and a number of additional convenient design tables have been introduced. The additions total 700 pages.

The discussion of general principles of economics and design illustrated by examples of Current Practice in Part I has been made more thorough as it is our experience that if the younger men have a good understanding of broad general principles they can more readily apply the detail rules to specific problems.

The general character of the second part of the book remains unchanged; namely a concise collection of data ordinarily required for everyday work.

For the convenience of users the book is bound either complete in one volume for office men or students or in two volumes for field men which gives a handy pocket thickness for the volume containing the field data. A full table of contents as well as the regulation index adds to the ease of locating desired information.

Readers are requested to suggest additions or changes in style of presentation which will increase the value of the book. The present edition is the outgrowth of such suggestions.

E. A. Bonney has supplied the data on Contractors Equipment. W. G. Harger has handled the balance of the revision.

W. G. Harger.
E. A. Bonney.

Rochester, N. Y., September, 1927.

PREFACE TO THIRD EDITION

THE present revision was undertaken in response to the suggestions and requests of many users of the earlier editions. The practical value of the Handbook is increased by the addition of approximately 350 pages of new material covering mountain road location and design, camp equipment, medical notes, notes on photography, the selected soil and gravel treatment of moderate traffic roads, and the more recent developments of hard surfaced types. There is no change in the general scheme of the publication, which is primarily a compact collection of reference data and time saving tables. For the benefit of men not entirely familiar with the road problem, the discussion of principles has been retained, and in some cases where it has been shown that certain arguments in the previous editions have failed to make the impression warranted by their importance, the discussion has been amplified and illustrated by examples of construction and design. We wish particularly to emphasize gradeline design, which is not at present receiving the attention to which it is entitled, and also point out the practically universal lack of adequate maintenance.

The costs given in the body of the text are for comparative purposes only and are based on labor at from \$0.175 to \$0.20 per hour and material costs of the period 1912 to 1915.

For the improvement of future editions we request your coöperation in the correction of typographical errors, and the addition of any omitted data generally useful in road work.

Very few highway engineers are satisfied with the road legislation or technical practice of today or believe that it can be applied as it stands to solve the highway problem in this country in the next fifty years, but the data that has been collected from experience serves as a basis for future improvement. There is every reason to be optimistic in regard to road development provided the problem is approached with constructive imagination and encouragement is given to departure from methods whose main defense lies in precedent or habit.

The work of revision for this edition is entirely that of W. G. Harger.

W. G. H.
E. A. B.

ROCHESTER, N. Y., January, 1919.

PREFACE TO SECOND EDITION

SINCE the publication of the first edition of this book four years ago, considerable progress has been made in the practice of road design and construction. To meet this advance, thi handbook has been revised by bringing the material on top courses up-to-date, and by adding considerable data on tests, designs, costs, maintenance and specifications. Not only has much of the old material been revised, but new material, totaling approximately 100 pages, has been added. The criticisms and suggestions of many who have used the book in the field and office have aided the authors in this revision.

A more complete and systematic index has been prepared by Mr. Percy Waller.

The general arrangement of the book remains untouched.

W. G. H.
E. A. B.

ROCHESTER, N. Y., May, 1916.

PREFACE TO FIRST EDITION

THE purpose of this book is to collect, in a compact and convenient form, information ordinarily required in the field and office practice of road design and construction.

The book is designed to meet the requirements of both experienced and inexperienced road men. The material on the relative importance of the different parts of the design, and the possibilities of economy, without impairing the efficiency of the road, are primarily for the inexperienced engineer. The collection of cost data and the tables will be useful to any one engaged in road work.

As it is difficult to avoid clerical errors and mistakes in proofreading in first editions, we shall appreciate the coöperation of readers in calling our attention to any errors.

W. G. H.
E. A. B.

ROCHESTER, N. Y., April, 1912.

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HANDBOOK

FOR

HIGHWAY ENGINEERS

PART I—PRINCIPLES AND PRACTICE

CHAPTER I

GENERAL ADMINISTRATIVE, ENGINEERING, AND ECONOMIC DATA

Introduction.—There is no object in giving an extended discussion of the general highway problem in a handbook of this character. The reader is referred to the many excellent highway books and official government publications for such a discussion. This chapter summarizes briefly the main points to be considered and tabulates basic administrative and economic data.

Public business as well as private business is usually based on the fundamental idea of creating a demand for commodities or services in order that large sums of money may be collected and its expenditure controlled by the constituted authorities. In order to produce a healthy and long-lived business, public or otherwise, it is necessary to follow a line of procedure which is both popular and economically defensible and which raises the living standards of the community and increases its productive capacity. The highway executive must decide what will be popular and at the same time good business. He must distribute rewards for large public expenditures on a reasonable basis between the capitalistic or banker class, by some use of bonds and other interest-bearing methods, and the laboring and active business interests, by utilizing most of his expenditures for human labor and the development of actively productive business enterprises of road material and equipment manufacture.

In the pursuit of the universal and laudable object of public works, highway executives are between two fires: the desire of constituents for perfection in travel comfort, and a reasonable type of road improvement, taking into consideration the economics of highway service and cost. General policy must usually be a compromise between business utility standards and the natural desire of all individuals and communities to get as much pleasure out of existence as they can regardless of cost or regardless of the business wisdom of expenditure for luxuries. The main value of highway economics lies in providing a basis for judgment considering the problem from a business standpoint. *The practical use of economics* *is largely confined to providing a check on extravagance, as the desire of communities for excellent roads has much more influence on general administrative policy than the economic point of view.*

Successful general policy must be flexible in order to cover the wide range of traffic requirements from cheap earth development roads in pioneer districts to the heavy-traffic boulevards of metropolitan districts, and requires careful consideration of the relative cost and value of the different stages of highway improvement. Complete highway programs include three well-defined branches coordinated by centralized administration:

1. The immediate temporary improvement of portions of the system not yet reached by the modern construction program.
2. The gradual improvement of the road system by means of economic locations, modern pavements and bridges suitable for the traffic each road must carry, and the proper care of detours during construction.
3. The maintenance and renewal of such high-grade improvements.

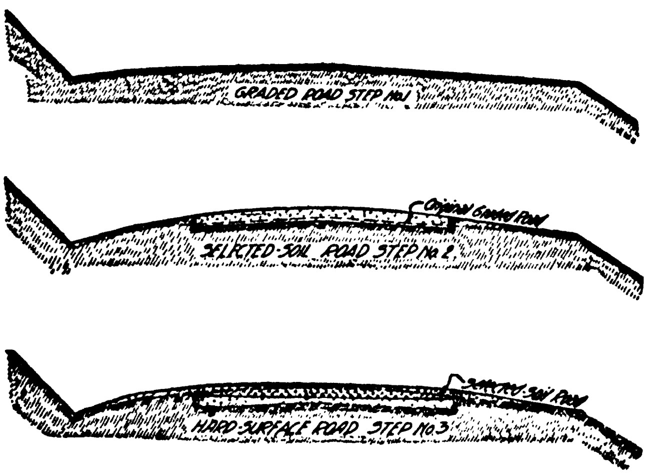


FIG. 1.—North Carolina progressive improvement.

As available funds are rarely sufficient to meet the final desires or needs of the community, it has been found that the policy of "progressive improvement" is a good practical solution, as each stage of improvement materially increases the comfort and safety of traffic and reduces its cost as compared with previous conditions, which retains the continued support of taxpayers for the gradually increasing costs during the period of years necessary for the completion of a satisfactory and economically defensible system of roads. Progressive improvement recognizes the two main stages of highway construction listed in the preceding paragraph: It emphasizes the well-established principle that the first necessity is the rapid construction of a complete system of moderate-priced roads which will give the community year-round service for its ordinary business needs, and that after this is accomplished it is desirable to raise gradually the standards of comfort and convenience to the highest point which the community can afford.

Reasonable administrative policy and engineering design give careful consideration to the following factors, which are discussed briefly:

1. General value and cost of highways (see p. 3).
2. Distribution of taxation (see p. 17).
3. Financing (see p. 19).
4. Traffic requirements (see p. 26).
5. Traffic regulation (see p. 28).
6. Traffic volume (see p. 28).
7. Traffic safety and speed (see p. 33).
8. Traffic range (see p. 38).
9. General suitability of pavement types (see p. 40).
10. Classification of roads (see p. 46).
11. Importance of maintenance (see p. 48).
12. Departmental organization (see p. 49).
13. Contract relations (see p. 51).
14. Economic tests of detail design (see p. 52).

1. General Value and Cost of Highway Improvements.—Highway value consists of two factors, first the indirect general benefits expressed in terms of higher standards of living, greater power of communication, and better general values as listed in Table 1, and, second, the direct reduction in cost of vehicle operation.

In order to insure a real profit on investment, highway expenditures should have some reasonable relation to the value of the proposed improvements, and analyses of the indirect and direct values of highways are well worth while in arriving at general limits of expenditure. In some cases these analyses indicate that increased expenditures are desirable, and in some cases that proposed expenditures exceed rational amounts. As a general rule, permissible increases apply only to main roads, carrying over 1500 vehicles daily, and reductions to moderate-traffic roads carrying less than 1000 daily. For roads carrying less than 300 to 500 vehicles daily, the general intangible benefit usually controls both minimum and maximum expenditures. For roads carrying more than 500 to 1000 vehicles daily, the direct reduction in travel cost controls permissible maximum expenditures, which in most cases permit economic design; on these roads general intangible benefit fixes the extreme minimum of expenditures (see Fig. 3, p. 16). There is no doubt that rational highway expenditures are a sound business investment for all communities, but in many cases the general public has been gradually educated up to high-type roads and extreme comfort of travel without any real understanding of the economics of the matter, and the present insistent demands for excessive expenditure on moderate-traffic roads are somewhat disturbing. It is believed that it is desirable to set reasonable economic limits for such cases. While needless expenditures are not economically defensible, it is quite obvious that highways cannot be considered solely from an economic standpoint and that analyses of this nature are only one phase of the problem.

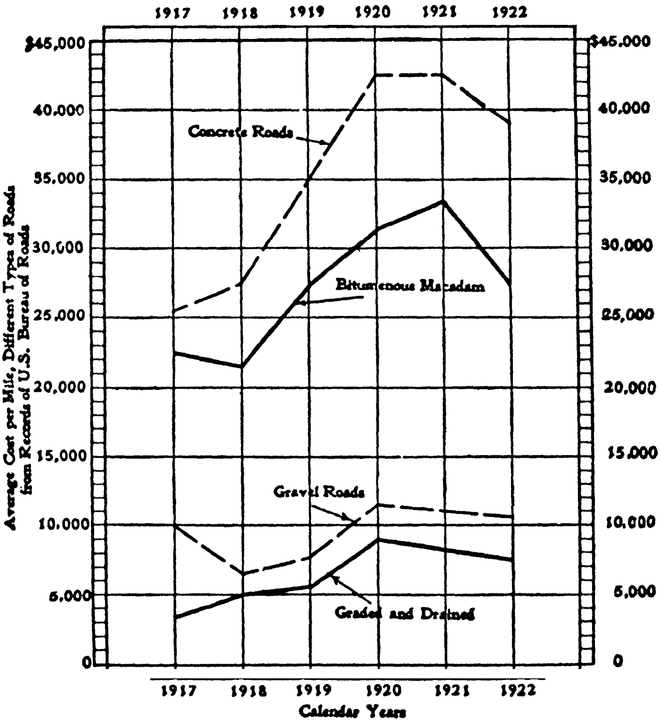
TABLE 1.—INDIRECT VALUE OF HIGHWAYS

1. They provide lines of communication to all parts of the district and save time of travel.
2. They provide direct-contact hauling service to all parts of the producing area of the district for the entire year.
3. They make rural life more attractive and tend to stabilize a healthy ratio of rural and city population.
4. They increase the social and recreational possibilities of both city and rural residence.
5. They increase the flexibility and the strength of the general transportation system of the county in times of unusual stress, such as military use in time of war and emergency transportation in case the railroad systems are tied up by strikes or otherwise.
6. They raise rural land values.
7. They increase the range of marketing in respect to both distance and favorable selling periods.
8. They promote suburban residence of city dwellers and reduce the necessity for congested living conditions.

Indirect General Benefits.—While it is impossible to put an exact money value on the indirect general benefits, it is certain that they are much more important than the item of reduced vehicle operation cost. It is also certain that the indirect benefits alone more than justify the construction of a complete system of modern highways usable the year round for moderate loads with moderate comfort and safety. Roads of this class will be termed "general-utility highways." Their value consists primarily in complete mileage and in their ability to handle the normal business and pleasure traffic of the community the year round. This value is not directly proportional to the volume of traffic, nor does it depend on extreme refinements of pavement surface, grades, and alignment in order to give perfection in travel comfort and to reduce motor operation costs to a minimum. These highways reduce motor operation costs in a varying degree, depending on the type of pavement and on the grade reductions necessary for general-utility purposes. No attempt should be made to reduce the cost of vehicle operation below that obtained by the normal utility standards for construction or reconstruction programs financed by general tax levies. Where such programs are financed by direct vehicle taxation, increased standards of construction may be justified by the increased direct saving to traffic, but even under such programs it is desirable to scrutinize the relation of proposed cost to value and to limit improvements over and above utility standards to amounts justified by actual traffic money savings.

The alignment, grading, bridges, safety provisions, and type and width of pavements required to fulfil the functions of general-utility roads vary for different volumes of traffic and for the cost and availability of different kinds of road materials, and should be

TYPICAL HIGHWAY COSTS



selected so that the final yearly cost of the highways is kept to a minimum. This cost includes interest on construction investment, maintenance, and renewal charges.

Table 2 summarizes the requirements for general-utility highways serving different volumes of traffic. This table has a quite universal value, as it is based on the experience of engineers over a long period.

TABLE 2.—GENERAL SLIDING SCALE OF MINIMUM SPECIFICATIONS TO MEET TRAFFIC NECESSITIES

Item	Pioneer roads Class IV, less than 100 vehicles daily year-round average	Agricultural roads Class IV, less than 300 vehicles daily average	General-utility roads — Class III, 300–800 vehicles daily average	Detail data index page No.
Right of way	100′	50′ minimum	60′ minimum	177
Culverts:				
Type	Semipermanent	Semipermanent	Permanent	212
Loads	H-13-H-15	H-13-H-15	H-15	200
Roadway width	Normal	Normal	Normal	203
Bridges:				
Type	Semipermanent	Permanent	Permanent	205
Loads	H-13-H-15	H-13-H-15	H-15	200
Roadway width, feet	12–18	20	22	203
Bridge sidewalks	None	None	None	203
Grades:				
Maximum	5–10%	5–8%	5–8%	105
Minimum	0 3%	Level	Level	109
Intermediate	No reduction	No reduction	No reduction	106
Compensation for alignment	3% on 40′ radius; 4% on 80′ radius	Compensate	Compensate	118
Alignment:				
Normal maximum curvature	100′ radius (40′ radius minimum)	250′ radius	500′ radius	120
Minimum at right-angle curves (in open country)	100′	100–200′ radius	250′ radius	120
Minimum on steep grades (5% or steeper)	100′ radius	300–400′ radius	500–600′ radius	120
Roadway sections:				
Width grading, feet	10–30	20–30	22–36	155
Width pavement, feet*	8–12	8–12	15–18	130
Width hard shoulders	No special provision	No special provision	18–20	130
Turnouts	Provided	None	None	164

Pavements:				
General types	Natural or selected soil	Selected soil, sand clay-gravel or macadam	Gravel or macadam	40
Safety provisions:				
Sight distance, feet	No limit	250	300	114
Guard rail	Some wooden rail	Wooden rail	Concrete posts or cable rail	676
Banked curves	Needed	Needed	Needed	124
Pavement marking (traffic lanes)	None	None	None	698
Railway crossing protection	Warning signs	Warning signs	Automatic signals	599
Pedestrian traffic	No provision	No provision	No provision	177
Danger signs	Provided	Provided	Provided	687
Direction signs	Provided	Provided	Provided	687
Illumination	None	None	None	699
Approximate Typical Costs per Mile of Road (1925 Cost Conditions) (Including bridges and grade crossing protection as listed above)				
Typical costs per mile for original construction	$1,000–$25,000	$10,000–$15,000	$15,000–$30,000	16
Approximate yearly cost of maintenance plus approximate yearly cost of renewal	20– 200	600 ±	1,400 ±	66
Total approximate yearly costs including interest on first cost maintenance and renewal	100– 1,400	1,200 ±	2,500 ±	66
Approximate yearly saving to traffic over earth-road conditions per vehicle mile	Development roads, impossible to figure	0.01 ±	0.015 ±	14
Approximate yearly saving to traffic per mile of road for average volume different classes of traffic	Development roads, impossible to figure	700 ±	3,200 ±	

* NOTE.—These widths are for straight alignment. For extra width on curves see page 132.

TABLE 2—*Continued*

Item	General-utility roads		Main rural roads, metropolitan districts, over 6000 vehicles daily	Detail data index page No.
	Class II, 800–2000 vehicles daily average	Class I, 2000–6000 vehicles daily average		
Right of way	70′ minimum	80′ minimum	80–100′ minimum	177
Culverts:				
Type	Permanent	Permanent	Permanent	212
Loads	H-15	H-20	H-20	200
Roadway width	Normal	Normal	Normal	203
Bridges:				
Type	Permanent	Permanent	Permanent	205
Loads	H-15	H-20	H-20	200
Roadway width, feet	22–24	24–30	30–40	203
Bridge sidewalks	As needed	Sidewalk	Sidewalk	203
Grades:				
Maximum	5–7%	5–6%	5%	105
Minimum	Level	Level	Level	109
Intermediate	No reduction below 4%	No reduction below 3%	No reduction below 3%	106
Compensation for alignment	Compensate	Compensate	Compensate	118
Alignment:				
Normal maximum curvature	500′ radius	600′ radius	800′ radius	120
Minimum at right-angle curves in open country	300′ radius	400′ radius	400′ radius	120
Minimum on steep grades (5% or steeper)	500–600′ radius	600–800′ radius	800–1,000′ radius	120
Roadway sections:				
Width grading, feet	24–36	32–40	45–54	156
Width pavement, feet*	16–18	18–20	27–36	130
Width hard shoulders	20–24	22–27	36–45	130
Turnouts	None	None	None	164

Pavements:				
General types	Macadam or rigid types	Rigid types	Rigid types	40
Safety provisions:				
Sight distance, feet	350	350	400	114
Guard rail	Concrete posts or cable rail	Cable rail or solid wall	Solid wall	676
Banked curves	Provided	Provided	Provided	124
Pavement marking (traffic lanes)	As needed	Provided	Provided	698
Railway crossing protection	Automatic signals or eliminations	Eliminations	Eliminations	599
Pedestrian traffic	As needed	Sidepaths	Sidewalks	177
Danger signs	Provided	Provided	Provided	687
Direction signs	Provided	Provided	Provided	687
Illumination	None	As needed	Provided	699

Approximate Typical Costs per Mile of Road (1925 Cost Conditions)
(Including bridges and grade crossing protection as listed above)

Typical costs per mile for original construction	$30,000–$50,000	$70,000–$90,000	$140,000 ±	16
Approximate yearly cost of maintenance plus approximate yearly cost of renewal	2,000 ±	3,000 ±	4,000 ±	66
Total approximate yearly costs including interest on first cost maintenance and renewal	4,000 ±	6,600 ±	11,000 ±	66
Approximate yearly saving to traffic over earth-road conditions per vehicle mile	0.017 ±	0.02 ±	0.02 ±	14
Approximate yearly saving to traffic per mile of road for average volume different classes of traffic	7,500 ±	28,000 ±	60,000 ±	

* **Note.—These widths are for straight alignment.** **See page 132 for extra width on curves.**

TABLE 3.—APPROXIMATE NECESSARY CONSTRUCTION COSTS GENERAL UTILITY HIGHWAYS (WESTERN NEW YORK)
(1925 cost conditions)

Approximate average number of vehicles per day (24 hr.), average mixed traffic	Approximate average construction cost per mile		
	Grading, culverts, pavement, and minor incidentals	Bridges over 5′ span	Railroad grade-crossing eliminations, one every 5 miles
0– 300	$ 5,000–$15,000	$ 3,000 ±	15,000 ±
300– 800	15,000– 30,000	4,000 ±	20,000 ±
800–2,000	30,000– 50,000	5,000 ±	20,000 ±
2,000–6,000	50,000– 70,000	6,000 ±	25,000 ±

Table 3 costs include grading, drainage, and pavement. For the ordinary road where the grade line follows the natural surface, the item of grading is largely necessary to provide a suitable roadbed for the pavement and is properly a pavement charge. Where hills are cut down and distance shortened, the economic value of such work (see Tables 5 and 6, p. 12) should be added to the values given in Table 7 before comparing with Table 3.

Table 3 summarizes typical average costs of a large mileage of general-utility highways under different volumes of traffic in western New York for 1925 cost conditions. Grade-crossing eliminations are not strictly an integral part of this type of highway and are discussed as a special problem in Chap IX. Table 3 will vary for different localities depending on soil conditions, local road materials, necessary bridge replacements,[1] and local contract costs. Tabulations of this general character are the first necessary step in the rational comparison of highway costs and value for any specified district.

Direct Reduction in Cost of Vehicle Operation.—The service value of highways in respect to reduction in the cost of vehicle operation depends on the volume and character of the traffic and on the reduction in cost of operation per vehicle mile. All of these factors are subject to considerable uncertainty, but the data now at hand make it possible to arrive at reasonable general conclusions, although they are not yet sufficiently definite to warrant hair-splitting decisions upon the details of design.

Methods of estimating present and probable future traffic are discussed in detail under Traffic Volume (p. 32). For this part of the discussion it is necessary to consider only the relation of business to pleasure traffic in arriving at reasonable allowances for highway value to the community at large and to individual traffic operations. The very complete Connecticut Traffic Census taken under the joint cooperation of the state and the U. S. Bureau of Public Roads in 1922 showed that 65% of the passenger autos can be classed as pleasure traffic and 35% as business traffic. Western New York investigations indicate that about 50% of the vehicles on the main state road system are pleasure and 50% business traffic. As the community at large derives no direct tangible business benefits

[1] Bridge contracts for the United States, 1920 to 1925, amount to 15% of cost of road and street contracts.

from the reduction of motor operating costs on private pleasure trips, the author has been in the habit of using one-half of the total traffic counts in computing the money saving of improved highways to the community at large and the total traffic in computing the money value of highways to vehicle owners.

This means that for general-utility highway construction financed by general tax levy it is probably reasonable to figure only one-half of the justifiable expenditure that can be figured for reconstruction boulevard construction financed by direct vehicle taxation.

Detail motor operation costs are given in Chap. XV, Cost Data. Table 4 (p. 11) gives a rough approximation of the relative cost of motor operation on different types of road improvements. Table 5 gives an approximation of the relative capitalized cost of motor operation on different grades. Table 6 gives the same data modified for business operation only. The total value of improvements includes all reductions in motor operation costs due to shorter distance, reduced grades, and a better type of pavement.

(text continued on page 14.)

TABLE 4.—RELATIVE COSTS OF VEHICLE OPERATION ON VARIOUS CLASSES OF ROADWAY SURFACES[1]

Type of surface	Type and speed of vehicle			
	Solid-tire trucks, 10 m.p.h.	Pneumatic-tire trucks, 15 m.p.h.	Automobiles, 25–35 m.p.h.	Motor busses, 25 m.p.h.
	Cents per ton-mile	Cents per ton-mile	Cents per vehicle mile	Cents per ton-mile
Average Portland cement concrete and asphalt-filled brick	8.00	8.30	10.00	24.00
Best Portland cement concrete and asphalt-filled brick	7.75	7.70	9.30	22.50
Best gravel, yearly average	8.50	8.80	10.90	25.70
Ordinary gravel, yearly average	9.00	9.40	11.80	27.80
Water-bound macadam, well maintained	8.70	8.95	11.10	26.00
Bituminous macadam, well maintained	8.50	8.80	10.60	25.70
Average sheet asphalt, yearly average temperature	8.10	8.30	10.00	24.00
Average asphaltic concrete, yearly average temperature	8.00	8.30	10.00	24.00
Best earth, well packed by traffic, yearly average	9.20	9.50	12.00	27.80
Ordinary earth with light traffic, yearly average	9.50	9.95	12.60	29.60

[1] AGG, T. R., and CARTER, H. S., in cooperation with U. S. Bureau of Public Roads, *Bull.* 69, Iowa State College, July, 1924. See also p. 68 for discussion of operating costs on different surfaces and the capitalized value of reductions in vehicle operation costs by different kinds of high-class pavement construction.

TABLE 5.—APPROXIMATE CAPITALIZED COST AT 5% OF OPERATING 100 MOTOR VEHICLES DAILY (36,500 PER YEAR) FOR 1′ OF DISTANCE ON THE VARIOUS GRADES[1]
(Time factor included)

Rate of grade	1 Average traffic trucks and light vehicles, based on 11 cts. per mile average grades	2 Light motor vehicles, based on 8 cts. per mile average grades	3 Heavy commercial trucks, based on 40 cts. per mile average grades
Level	$14.80	$10.70	$54.00
1	14.80	10.75	54.00
2	15.10	10.85	54.60
3	16.10	11.05	56.70
4	17.90	11.40	63 90
5	20.50	12.00	74.10
6	23.30	12.90	84.50
7	26.40	14.50	95.00
8	29.60	16.15	105.60
9	32 80	17.90	116.20
10	36.00	19.70	127.00

NOTE.—This table to be used only for rough general comparisons of the relative value of alternate long routes. The actual costs must be used with caution. Alignment good. For effect of dangerous alignment, see page 118.

[1] "Highway Location, Grading and Drainage," McGraw-Hill Book Company, Inc.

TABLE 6.—APPROXIMATE CAPITALIZED COST AT 5% OF OPERATING 100 MOTOR VEHICLES DAILY (36,500 PER YEAR) FOR 1′ OF DISTANCE ON THE VARIOUS GRADES[1]
(Time factor modified columns 1 and 2. Fuel cost not considered for 50% of light motors columns 1 and 2)[2]

Rate of grade	1 Local service roads and secondary state routes, less than 1800 vehicles daily average	2 Main intercity state roads, probable future traffic 1800 or more daily average	3 Future commercial truck to 11 roads
Level	$ 9.10	$12.20	$ 54.00
1	9.10	12.20	54.00
2	9.15	12.30	54.60
3	9.25	12.50	56.70
4	9.40	12.80	63.90
5	9.70	13.50	74.10
6	10.20	14.50	84.50
7	11.20	15.80	95.00
8	12.50	17.70	105.60
9	14.00	19.80	116.20
10	15.50	22.00	127.00

NOTE.—As a general rule, make no changes in existing locations or grade lines which fit the natural surface unless the saving in approximate operating cost is at least twice the additional cost of construction. This note does not apply to changes which are desirable to improve the safety or convenience of the road.

[1] "Highway Location, Grading and Drainage," McGraw-Hill Book Company, Inc. This table serves as a practical basis of comparison of alternate routes or alternate grading designs. (See p. 57 for a detailed tabulation showing the method of using these data.)

[2] U. S. Bureau of Public Roads Traffic Census in Connecticut, 1922, showed that 65 % of the passenger autos were pleasure vehicles and 35 % business vehicles.

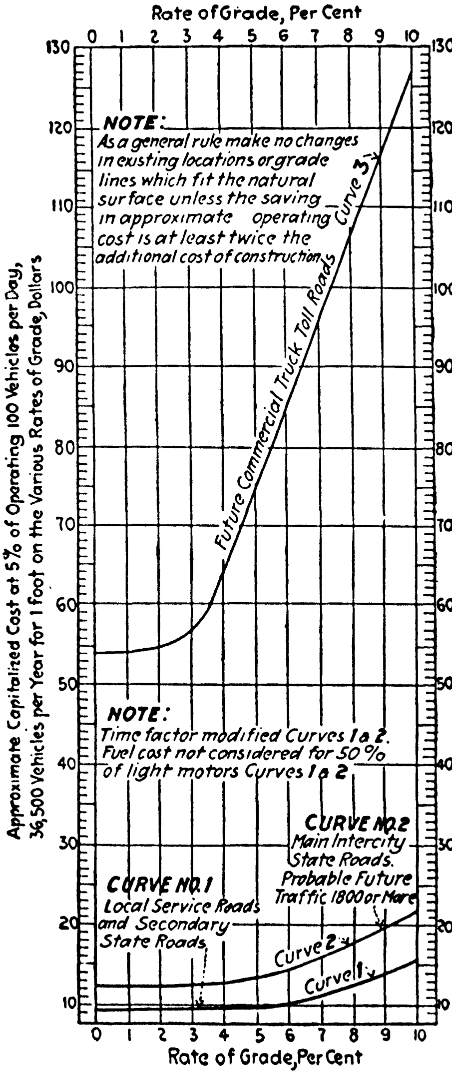


FIG. 2.—Graph of Table 6. A practical basis of comparison of alternate routes or alternate grading designs. This graph assumes one-half of traffic uphill and one-half downhill.

Figures of this kind must, of course, be used with caution. They do not consider the effect of snow and ice for a portion of the year, nor do they consider the gradual deterioration of the higher-type surfaces which tend to reduce the apparent advantage of the higher types. Table 4 also shows considerably more difference between well-maintained modern macadams and rigid pavements than the author's experience indicates (see p. 68). Considering all these factors and the facts that the more important roads are better located, better graded, better maintained, and are kept free of snow in winter, the author has been in the habit of using the following values for saving in average vehicle operation per mile for different classes of general-utility road improvements as listed in Table 2 (p. 6) over and above earth-road conditions.

CLASS OF ROAD

	Class I, over 2,000 vehicles daily, in cents	Class II, 800 to 2,000 vehicles daily, in cents	Class III, 300 to 800 vehicles daily, in cents	Class IV, less than 300 vehicles daily, in cents
Total value to vehicle owners	2 per vehicle mile	1.7 per vehicle mile	1.5 per vehicle mile	1 per vehicle mile
Approximate value to community at large	1	0.8	0.7	0.5

If justifiable road cost is considered strictly from the standpoint of reduction in vehicle operation cost, the above savings per year per mile of road must at least pay for the yearly maintenance, renewal, and interest charges of the highway improvement.

Anticipating the discussion of maintenance and renewal, the yearly road cost for interest on construction, maintenance, and renewal depends largely on the volume of traffic and is only slightly affected by the type of pavement, provided the pavements used are of the general class indicated in Table 2. This yearly charge ranges from 7 to 12% of original cost of the improvement for general-utility highways and averages about 10%. This, of course, is subject to some variation, but is not misleading for purposes of arriving at the extreme justifiable maximum expenditure for construction for a large mileage of improved roads. This maximum justifiable expenditure is, therefore, approximately 10 times the resultant saving to traffic from the improvement. If the community desires any real return on its investment it is well not to exceed two-thirds of the maximum amount. Table 7 is compiled on this basis, assuming a saving of 1 ct. per vehicle mile.

TABLE 7.—ALLOWABLE COSTS
(1921 cost conditions)

Average daily number of vehicles	Assumed yearly travel cost saving, based on 1 ct. per vehicle mile, average mixed traffic	Maximum allowable expenditure per mile for original construction, based on pavement benefit	Reasonable expenditure with some profit to the community
100	$ 350	$ 3,500	$ 2,300
200	700	7,000	4,600
300	1,050	10,500	6,900
400	1,400	14,000	9,200
500	1,750	17,500	11,500
600	2,100	21,000	13,800
700	2,450	24,500	16,100
800	2,800	28,000	18,400
900	3,150	31,500	20,700
1,000	3,500	35,000	23,000
1,500	5,250	52,500	34,500
2,000	7,000	70,000	46,000
3,000	10,500	105,000	69,000
4,000	14,000	140,000	92,000
5,000	17,500	175,000	115,000
10,000	35,000	350,000	230,000

Column 3 represents maximum allowable expenditure. If construction equals this amount, the community gets no money return on their investment; they merely take their money out of one pocket and put it in the other. It is, therefore, not desirable to use more than two-thirds of these amounts if a profit is considered desirable; column 4 has been figured on this basis.

Figure 3 compares graphically the necessary and allowable construction costs per mile for general-utility highways based on a saving of 1 ct. per average vehicle mile. Comparisons of this kind, making due allowance for the degree of improvement accomplished and the method of financing, usually indicate the following general conclusions:

1. On roads carrying an exceptionally heavy traffic, the actual travel saving will more than justify high-grade types of construction.

2. For moderate traffic (say, 300 to 1500 average vehicles per day) satisfactory general-utility highways are justified, but needless refinements may well be avoided if any real return on investment is desired.

3. For purely local roads carrying less than, say, 300 to 500 vehicles per day it is probably necessary to consider the intangible benefits to justify a satisfactory standard of construction. This is shown in chart form on page 16 and begins to help in the matter of tax burden distribution.

Summarizing the limitations imposed on expenditures by the value of highways: If Fig. 3 is modified slightly to fit actual observed road service, it is evident that if construction investment is considered from the standpoint of direct return by reduction in cost of vehicle operation, we are limited to the standard of general utility except for heavy-traffic roads carrying an average volume of over, say, 1500 vehicles per day. This class of road represents a very small percentage of the total road mileage. Even for roads carrying a year-round average of more than 1500 vehicles per day, there is no particular advantage in adding much to the amounts shown by the general-utility curve.

If construction investment is considered from the standpoint of intangible benefits expressed in terms of convenience, safety,

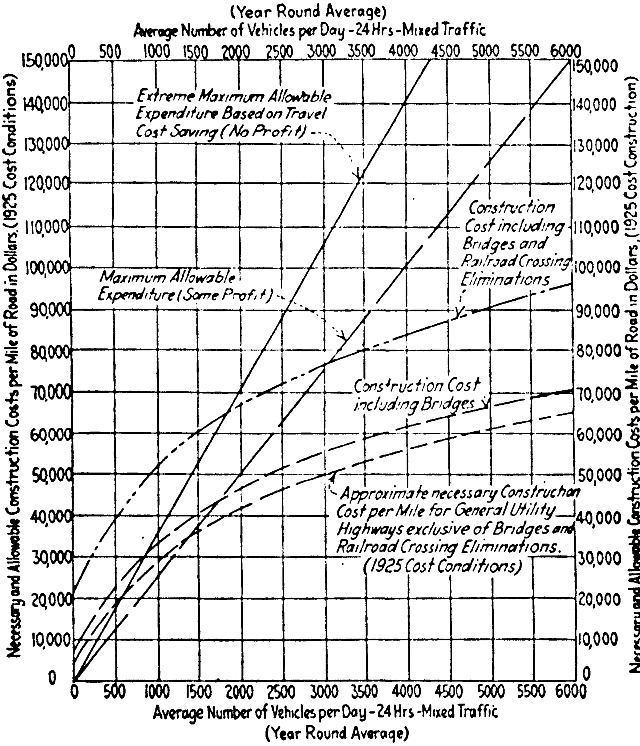


FIG. 3.—Comparison of approximate necessary and allowable construction costs for general utility roads in Western New York under different volumes of traffic. (Construction costs for 1925 conditions. Allowable costs based on 1 ct. per vehicle mile benefit.)

pleasure, and ease of communication, no practical limitation of expenditure except that imposed by the community's financial ability to complete a general-utility system within a reasonable time need be considered.

The tax burden distribution also has a distinct bearing on the proper limitation of expenditure; that is, if the roads are being improved by means of a general tax levy it is desirable not to exceed the costs required by general-utility standards. If the road funds are raised by direct vehicle taxation more liberality in expenditures is justified for added convenience and pleasure.

2. Distribution of Tax Burden.—A successful tax is based on three main points:

1. The final burden should fall on each individual as nearly as possible in proportion to the direct and indirect benefit received.
2. The direct tax should be paid by the individuals receiving the immediate direct benefit if they are financially able.
3. The tax must be levied on a definite source comparatively easy to assess and be collected from individuals having the ready money for payment.

Real property, motor vehicles, tractor and horse vehicles, and gasoline are definite sources of taxation owned or used by individ. uals, presumably with ready cash to pay any reasonable tax burden-

The immediate and direct benefits of improved highways are largely received by the owners of vehicles operating on the roads.

The final direct and indirect benefits of a modern highway system were given on page 3.

While the road user gets most of the direct benefit, it is rarely feasible in the first stages of the improvement to tax him directly for a large proportion of the cost of construction; that is, before improved highways are an accomplished fact, the vehicles are fewer in number and operated by financially poorer individuals. The completion of a modern highway system increases the number and effectiveness of vehicles and adds to the ability and willingness of the owners to assume a larger tax burden. It is, therefore, generally believed that the original construction of such a modern general-utility system (under a proper classification based on reasonable regulation of traffic) should properly be paid for largely by a general tax levy, which is eventually quite evenly distributed over the community.

The cost of maintenance and the renewal of pavements depend on the volume and kind of traffic. The largest share of the benefit of keeping a road continually in excellent shape goes to the vehicle owner. There are two general classes of traffic: pleasure traffic and business traffic. If a pleasure vehicle is taxed a fair amount to cover the damage it does to the road, this is a luxury tax borne by the owner. If a business vehicle is taxed a fair amount to make the highway self-supporting, the charge is added to the other vehicle operating costs which go into the price charged to the consumer, and the public at large foots the bill. The principle of a vehicle tax for maintenance and renewal apparently has a sound basis in fairness. While there may be minor flaws in its application, there are the same flaws in the application of a tax principle which is not essentially sound, so that the small practical inequalities of the tax do not invalidate the soundness of the general proposition.

It is certain that if maintenance and renewal were paid by general tax levy pleasure traffic would escape its fair luxury tax and the business traffic, particularly heavy hauling, would have an unfair advantage in competition with other transportation methods which pay their own cost of track or waterway construction and maintenance.

Public-road tolls are not feasible except for a few special cases, as they restrict the free movement of traffic, so that some form of graduated vehicle license based on gross vehicle weight and type of tire supplemented by a gasoline tax seems the most reasonable form of tax for the upkeep and renewal of at least the main highways.

These general principles expressed in terms of definite policy are as follows:

It is quite generally recognized that the intangible benefits of a road system practically obligate the community at large to construct such roads up to the standards of general utility by means of a general tax levy. This can be accomplished without exceeding the bounds of reason on the basis of a fairly reasonable length of time for the construction program and the use of reasonable length of term of serial bonds for raising construction funds.

The direct and intangible benefits accruing to vehicles using the highways practically warrant putting upon the owners of vehicles the cost of maintaining and renewing at least the main roads of such a system after it has once been built. For the purely local roads which are not self-supporting due to light volume of traffic (see p. 15), it is not reasonable to tax vehicles for their entire maintenance and upkeep; that is, vehicle license fees plus gasoline tax should probably be figured on the basis of taking entire care of the main roads, aggregating not over 15 to 25% of the total road mileage, and for raising the standard of such roads beyond that of general utility. For the purely local roads a small percentage of the maintenance cost can be properly charged against traffic. This can be accomplished by a graduated license fee and gas tax not beyond the bounds of reason. For a completed system in a well-settled area, it is probable that an average yearly tax of from $25 to $50 per vehicle will produce the necessary funds. Considering the mileage run by the average car, 4000 to 7000 miles per year with a yearly gas consumption of from 400 to 600 gal. per registered vehicle and the variations in license fees increasing rapidly for commercial trucks, this would not be an excessive amount.

Average Cost of Maintenance and Renewal per Vehicle Mile
(Western New York conditions)

Class of traffic	Average daily number of vehicles	Average yearly cost road maintenance and renewal for vehicle mile, cents	Average cost for interest on original construction cost plus maintenance and renewal, cents
Class I	4,000	0.2	0.3
Class II	1,500	0.3	0.6
Class III	600	0.5	0.8
Class IV	200	0.7	1.4

The raising of funds for immediate temporary traffic service over the gaps in the existing improved road system may well be raised by a slight addition to the vehicle tax; this affords no difficulties, as the immediate return to traffic is so self-evident that road users generally are in favor of such procedure. This tax gradually decreases as the system is completed, balancing the normal increase in the regular maintenance tax.

While it seems reasonable to care for the maintenace upkeep of the purely local roads by a yearly general tax levy, this is the most difficult part of the program from a practical standpoint. In the richer well-settled states it is not probably an excessive burden, but for the poorer districts it is likely that some help from some other source will be necessary. In such districts it is likely that it will be necessary to get along for some time with comparatively unsatisfactory conditions on the purely local roads.

The essential fairness of the road tax depends on a proper equalization of value on the assessment roll and a well-thought-out graduated vehicle tax. It is desirable to avoid rapid fluctuations in the tax rate. This can be accomplished in the matter of vehicle licenses and gasoline tax by gradually increasing them from year to year as the improved roads grow in scope and service (see p. 21). It can be accomplished in the matter of general tax levy for construction by adopting a reasonable term of years for the construction program and issuing the serial bonds only as fast as actually needed for raising construction funds (see pp. 22 and 23).

In order to apply the foregoing principle of taxation definitely to any specific road program, it is necessary to determine the point in any policy of progressive road improvement where the roads have reached the limit properly chargeable to general taxation and beyond which limit further improvement is a strict traffic charge. This occurs when the general road system serves the majority of the individual users for ordinary business and local pleasure purposes as shown in Table 2 (p. 6), and when further improvement is for the added convenience of specially heavy commercial trucking on secondary roads or for the extreme convenience of pleasure traffic. To arrive at any reasonable definite conclusion, the general character and range of normal traffic must be considered (see p. 27 for the formulated conclusion).

3. **Financing.**—There are three general classes of highway expenditure which require different methods of financing:

1. Extensive programs involving comparatively permanent types of roads and bridges which are immediately needed in order to open up new territory or to handle ordinary business traffic economically are usually financed by the sale of serial or sinking-fund bonds.

2. Yearly maintenance or temporary makeshift improvements must be financed by yearly "pay as you go" tax budgets.

3. The reconstruction of fairly suitable existing highways up to more luxurious standards should usually be financed by yearly "pay as you go" tax budgets, although short-term bonds are sometimes allowable.

Construction funds for extensive programs are generally raised by serial or sinking-fund bonds. The "pay as you go" yearly appropriation policy is sometimes used for small programs, as it is the cheapest in final cost, but for an extensive program the bond

(text continued on page 24.)

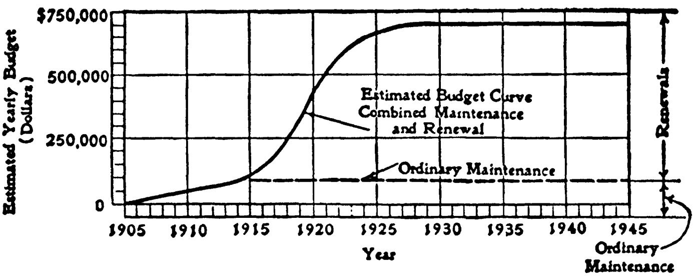


FIG. 19*A*.—Estimated yearly combined maintenance and renewal budget (1921 costs conditions).

Class I and II*A* roads (310 miles).

Assumed construction period 1905 to 1915.

Assumed life of pavement 12 to 15 years.

Note how the curve steepens up when pavement renewals becomes necessary and then flattens out to a high level line when the entire system is old enough for the renewal charge to become constant.

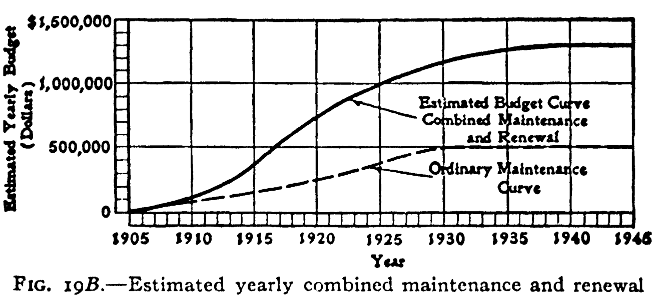


FIG. 19*B*.—Estimated yearly combined maintenance and renewal budget (1921 cost conditions).

Class II and III roads (1040 miles).

Assumed construction period 1905 to 1930.

Assumed life of pavement surface 8 to 12 years.

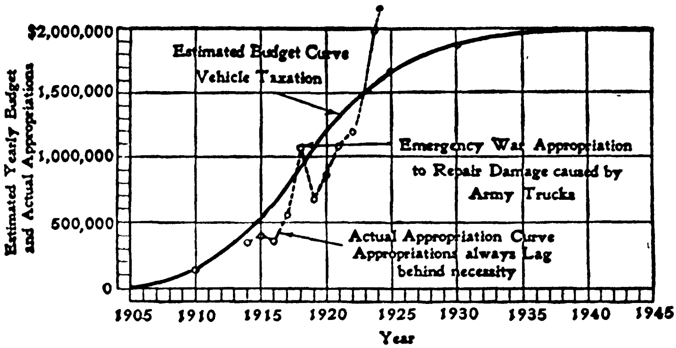


FIG. 19*C*.—This figure combines 19*A* and 19*B* and includes Classes I, II*A*, II and III roads.

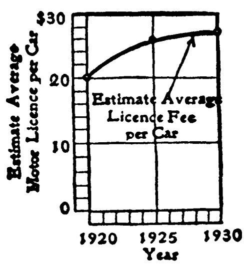
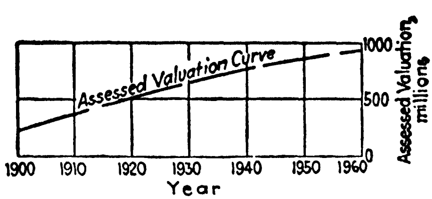


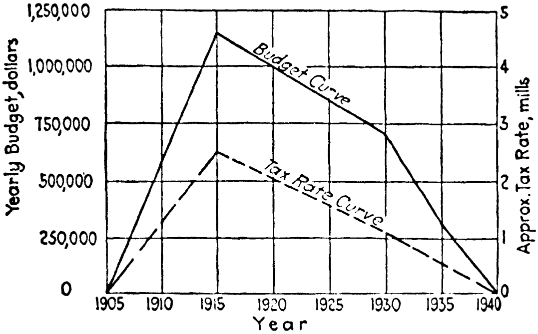
FIG. 19*D*.—Estimated necessary auto license fee.

FIG. 19.—Diagrams representing estimated combined maintenance and renewal yearly appropriations for District No. 4, Western New York.

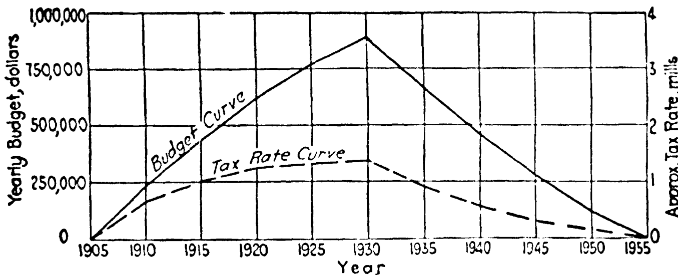
NOTE.—These appropriations should be raised by vehicle taxation. This data compiled in 1920 by W. G. Harger in connection with a personal study of Division No. 4 under assumed conditions to illustrate the value of classification in connection with a general finance plan. This same method can be applied to any program at any stage of procedure provided the data is carefully worked out considering the age and type of existing pavements.



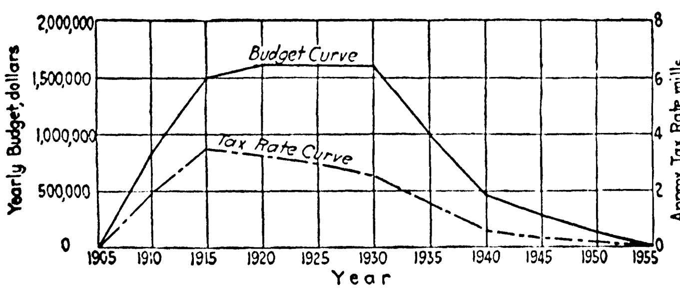
Estimated Assessed Valuation. Division 4.



Tax Curve for Construction Class I and IIA Roads. Division 4.



Tax Curve for Construction Class II and III Roads. Division 4.



Tax Curve Class I, IIA, II and III Roads. Construction Program

FIG. 20.

NOTE.—See page 25 for tabulation of budget.

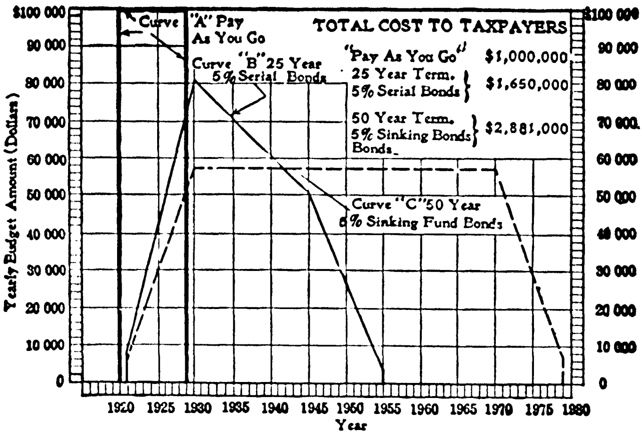


FIG. 4A.—Cost of different methods of financing.

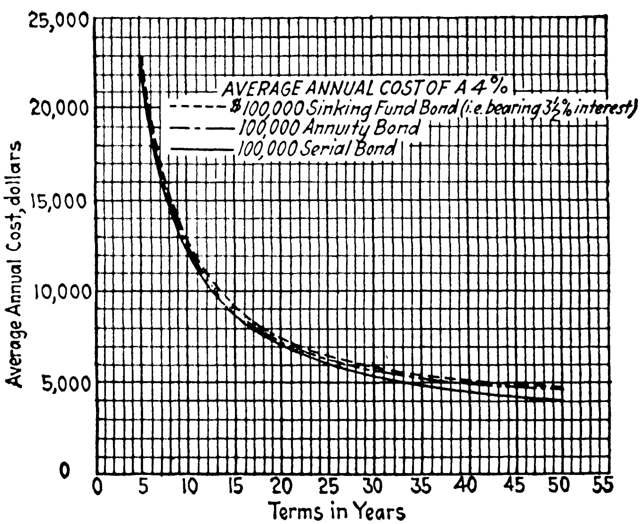


FIG. 4B.—Effect of term and kind of bond on average yearly repayment amounts.

method is to be preferred, as it permits more rapid construction, makes it possible to develop a more stable engineering organization, reduces rapid fluctuation of tax rates, and throws a fair share of the burden on to the future (for comparison of total and yearly cost, see Fig. 4, page 23).

Serial bonds are in more favor than the sinking-fund method, as the history of public sinking-fund bonds is not encouraging; the necessary yearly appropriations are often neglected and the sinking fund not well administered.

A reasonable term of bond depends on the rate of depreciation of the road improvement and on the maximum tax rate the community can afford.

An examination of Fig. 4*A* shows that the yearly tax budget charge for retiring bond issues does not lessen much for a bond term of more than 30 years. Considering the fact that the total cost of bond financing increases with the length of the term (see Table 7) and that the yearly charge does not decrease much for the longer terms, it is evidently desirable not to exceed the 25- or 30-year term. The 25-year serial bond method of raising funds for original improvement of highways seems to meet the requirements of the usual situation for the following reasons:

TABLE 7*A*.—TOTAL COST OF SINKING-FUND BOND

(Total cost of a $100,000 sinking-fund bond bearing 3, 4, 5, or 6% interest, with sinking fund drawing 3½%, and maturing at different periods from 5 to 50 years)

Terms in years	3 %	4 %	5 %	6 %
5	$108,241	$113,241	$118,241	$123,241
10	115,241	125,241	135,241	145,241
15	122,738	137,738	152,738	167,738
20	130,722	150,722	170,722	190,722
25	139,185	164,185	189,185	214,185
30	148,114	178,114	208,114	238,114
35	157,494	192,494	227,494	262,494
40	167,309	207,309	247,309	287,309
45	177,540	222,540	267,540	312,540
50	188,169	238,169	288,169	338,169

TOTAL COST OF SERIAL BOND

(Total cost of a $100,000 serial bond bearing 3, 4, 5, or 6% interest and maturing at different periods from 5 to 50 years)

Terms in years	3 %	4 %	5 %	6 %
5	$109,000	$112,000	$115,000	$118,000
10	116,500	122,000	127,500	133,000
15	124,000	132,000	140,000	148,000
20	131,500	142,000	152,500	163,000
25	139,000	152,000	165,000	178,000
30	146,500	162,000	177,500	193,000
35	154,000	172,000	190,000	208,000
40	161,500	182,000	202,500	223,000
45	169,000	192,000	215,000	238,000
50	176,500	202,000	227,500	253,000

The rate of depreciation of the different general kinds of modern highways is approximately as follows, assuming that the maintenance is moderately good and that the pavements are well designed and used under a volume of traffic for which they are suited (see Table 2, p. 6).

Road improvements include grading, drainage, pavement foundation, pavement surface course, and incidentals. The first three items are practically permanent. The surface and incidentals require renewals at quite regular intervals. Double-track macadam surfaces cost about 40% of the total cost of original improve-

TABLE 8.—TABULATION ILLUSTRATING METHOD OF FIGURING YEARLY BUDGET. SERIAL BOND FINANCING

(25-year bonds 5% interest)

Year	Bonds issued to date	Outstanding indebtedness	Annual charges		
			Interest[1] 5 %	Repayment[2] account	Budget for year
1905	$ 1,400,000	$ 1,400,000			
1906	2,800,000	2,744,000	$ 70,000	$ 56,000	$ 126,000
1907	4,200,000	4,032,000	137,200	112,000	249,200
1908	5,600,000	5,264,000	201,600	168,000	369,600
1909	7,000,000	6,440,000	263,200	224,000	487,200
1910	8,400,000	7,560,000	322,000	280,000	602,000
1911	9,800,000	8,624,000	378,000	336,000	714,000
1912	11,200,000	9,632,000	431,200	392,000	823,200
1913	12,600,000	10,584,000	481,600	448,000	929,600
1914	14,000,000	11,480,000	529,200	504,000	1,033,200
1915	....	10,920,000	574,000	560,000	1,134,000
1916	....	10,360,000	546,000	560,000	1,106,000
1917	....	9,800,000	518,000	560,000	1,078,000
1918	....	9,240,000	490,000	560,000	1,050,000
1919	....	8,680,000	462,000	560,000	1,022,000
1920	....	8,120,000	434,000	560,000	994,000
1921	....	7,560,000	406,000	560,000	966,000
1922	....	7,000,000	378,000	560,000	938,000
1923	....	6,440,000	350,000	560,000	910,000
1924	....	5,880,000	322,000	560,000	882,000
1925	....	5,320,000	294,000	560,000	854,000
1926	....	4,760,000	266,000	560,000	826,000
1927	....	4,200,000	238,000	560,000	798,000
1928	....	3,640,000	210,000	560,000	770,000
1929	....	3,080,000	182,000	560,000	742,000
1930	....	2,520,000	154,000	560,000	714,000
1931	....	2,016,000	126,000	504,000	630,000
1932	....	1,568,000	100,800	448,000	548,800
1933	....	1,176,000	78,400	392,000	470,400
1934	....	840,000	58,800	336,000	394,800
1935	....	560,000	42,000	280,000	322,000
1936	....	336,000	28,000	224,000	252,000
1937	....	168,000	16,800	168,000	184,800
1938	....	52,000	2,600	112,000	114,600
1939	....	0	0	56,000	56,000
1940	....	0	0	0	0
	....	....	$9,091,400	$14,000,000	$23,091,400

[1] Interest on outstanding indebtedness.

[2] Repayment account is 1/25 of bonds issued to date until principle is paid. For a 30 year term, repayment would be 1/30 of bond till principle was paid.

ment and generally last about 10 years before renewal is necessary; that is, for this class of improvement it is fairly safe to figure on a natural depreciation of 40% of the original investment in, say, 10 years, after which very little further depreciation occurs, as the balance of the work retains its value quite indefinitely if properly maintained. For rigid pavements, the surface course needs renewing in about 15 years as a rule, and this item amounts to about 40 to 60% of the cost of original improvement; that is, the depreciation is about 50 to 60% in 15 years. For any general system of highway improvements composed of macadams and rigid pavements, a bond term which results in paying off 40% of the principal and interest in 10 years and 60% in 15 years is probably sound financing; the 25-year (or less) bond complies with this condition.

Bridges and grade-crossing eliminations permit a maximum 40-year bond period.

For reconstruction of surfaces and yearly maintenance the "pay as you go" method financed by direct vehicle taxation for the main highways and by direct general tax levy for the purely local roads seems reasonable. If bonds are used for a reconstruction program to renew the pavement surfaces the bond term should probably not exceed 15 years, but this method is hardly warranted except in unusual cases.

Table 8 illustrates detailed computation of yearly tax budget for serial bond financing.

For a more complete discussion of Bond Financing see Location, Grading and Drainage of Highways, McGraw-Hill Book Company, Inc.

4. **Traffic Requirements.**—The shift in general mode of transportation from horse to motor is about completed. Horse traffic will not entirely disappear, but it is not a large factor in determining road policy in many localities. There are today (1926) approximately 18,000,000 motor vehicles in the United States, of which approximately 2,000,000 are trucks from ¾- to 7-ton capacity.

Motor traffic can be divided into three general classes:

1. The passenger car.
2. Light trucks and passenger bus lines.
3. Heavy commercial hauling trucks.

1. The light passenger car represents the largest part of the volume of road traffic, probably at least 85%. This type of traffic desires all-the-year-round roads which can be traveled at high speed with comfort, safety, and convenience. This kind of traffic does not require extreme strength of pavements, as the vehicles are light in weight, but it does require a fairly good surface comparatively free from dust.

2. The second class of traffic, namely, light trucks up to 2½-ton capacity and passenger bus lines, desires about the same general service as the light passenger cars except that the item of extreme comfort is not so important. They require somewhat stronger pavements to handle their loads the year round, but this requirement is not at all excessive and can be easily obtained on all the roads of a general-utility system without too much cost. Truck

traffic of this nature constitutes approximately 10% of the total road traffic today (1926) and represents the class of hauling equipment preferred by the individual engaged in farming, small-town business, parcel delivery, and general-utility hauling. Available information indicates that for the usual business man, farmer, and the parcel delivery of even large corporations the light $\frac{3}{4}$- to $2\frac{1}{2}$-ton trucks with pneumatic-tire equipment to at least the 1-ton size and probably to the 2-ton size will be the most popular type for hauling. This is due to reasonable first cost for the truck, moderate upkeep, high speed, comfort, and general utility.

Passenger cars, these light trucks, and what small amount of horse traffic still remains constitute at least 90 to 95% of the total number of vehicles using the roads; if the needs of these two classes of traffic are satisfied, the problem of giving complete service to the community at large is practically solved. *We can probably afford to build all the roads to satisfy 95% of the users without any restrictions in regard to load or reasonable speeds which would annoy the individual in the selection of his car or truck. This can be done with moderate-priced general-utility roads* (Table 2, p. 6).

3. The class of vehicle that gives trouble is the heavy commercial truck, which aggregates probably not over 5% of the total rural road vehicles, although for special roads they may run as high as 50% of the total volume. They require extremely strong high-priced, rigid pavements. The use of such trucks for moderately long hauls in competition with rail shipments is undoubtedly of considerable value to the community at large. How much it is worth and how much consideration should be given to this type in deciding on a general road policy is open to argument. It is quite generally conceded that the heavy truck seems to be the logical and popular means of intercity transportation of certain commercial products for moderately long hauls in competition with rail shipment, say 100 miles; but by reason of their greater cost per mile of operation they are not suited to make a large number of indlividual deliveries in successful competition with the small truck. They are, however, usefully employed for certain industries on colective routes, running on certain well-defined hauling arteries with definite stops at which the local shipments are picked up. For collecting milk, for instance, certain main-line roads through the heart of the producing district with collection stations at the crossroads will serve the purpose satisfactorily and economically. It is probable that if provision is made for strong pavements on the main natural commercial routes which aggregate not over 1 to 10% of total road mileage depending on the general character of the district, the community will derive about all the benefit possible from this class of hauling.

A general-utility system of highways consisting of a small mileage of heavy-truck roads and a large mileage suitable for passenger cars and light trucks seems to offer the most return on an investment in highways, as it in effect, gives the community about all of the advantages that it can derive from the system without unjustifiably large expenditure. Such a system can be properly constructed by general taxation, but if the secondary roads are increased in cost for the bene-

fit of heavy trucking, this added cost is not properly a general community charge and such additional cost should be paid by a traffic tax.

5. Traffic Regulation.—If the unregulated use of 5- to 7½- or 10-ton trucks over all the highways is permitted at all seasons of the year, a slight reduction in total hauling cost might result, but it would increase the first cost and maintenance of a general system out of all proportion to the added benefit in reduced hauling cost. (For the effect of different maximum loads on construction cost see Chap. VI, p. 425.) It is evident that, in order to get anywhere with a general system of roads within a reasonable term of years, some restriction must be placed on heavy-truck hauling. These restrictions should not be made any more severe than necessary, but reasonable restrictions in regard to load, speed, and range of travel are recognized as necessary. Restrictions on load and speed have already been accomplished, but restriction on range of travel has not yet been generally accomplished.

Present restrictions on improved roads do not usually permit over a 28,000-lb. gross weight (including weight of truck) and a wheel pressure not exceeding 800 lb. per inch width of tire (see Chap. VI, p. 370); existing statutes tend to limit the speed of solid-tire trucks to from 12 to 15 m.p.h. It is probably desirable to go a step further and limit the use of heavy trucks to specified hauling routes, and to limit the use of even the 5-ton truck to special hauling routes except in the dry season of the year. Statutory restrictions of this nature have been adopted in some of the states and a temporary reduction in gross loading during the spring is quite common procedure, but it is often not well enforced.

6. Traffic Volume. *General Data.*—Administrative interest in traffic volume is largely confined to the decision as to whether growth in volume shall be handled by concentration on a few main roads of exceptional width and strength, or by distribution over parallel routes of lower individual carrying capacity. The answer to this question appears to be that distribution of traffic is usually the proper solution. The principle of traffic distribution seems sound, for if a specified volume of traffic is distributed by the use of two roads of moderate width and cost in place of one road of exceptional width and exceptional cost, more territory is reached directly; congestion and danger are reduced and alternate routes are provided during repair periods. The normal growth in the number of improved roads tends to take care of normal traffic growth. That is, at the present stage of highway improvements it is not likely that it is justifiable to build a pavement width of more than 18 to 20′ on single roads; it is probably better to parallel the road for traffic of over 6000 daily (10-hr. count in summer) rather than increase the pavement widths, except for short distances near large cities where an unavoidable concentration of traffic occurs.[1] 3 lane pavements (27′ width) are good up to 7000 to 9000 vehicles (10 hours in summer) and beyond that volume 4 lane (36′ to 40′) are desirable.

[1] Maximum permissible hourly peak load for 2 lane traffic (20 ft. pavements) without dangerous congestion about 1500 vehicles per hour. (Penn. observations 1926.) Making allowance for ratio of peak hour to 10 hour average this checks the text 10 hour limits.

These capacities for different widths assume reasonable driving comfort for average mixed traffic and are based on Western New York experience on rural highways. 15,000 to 20,000 vehicles have passed over a two lane highway in this district in 24 hours but it is dangerous, inconvenient and decidedly unpopular.

Traffic volume fluctuates and gradually increases. Before a road system is completed, there is no definite information on which to base probable future traffic, but it can be approximated fairly closely by a careful study of the district in comparison with other similar districts in which a road system has been completed. Total motor vehicle registration gives some basis for estimating probable volume on the main roads, as while it is not a sure index of the volume of rural travel, it is a very good index of probable

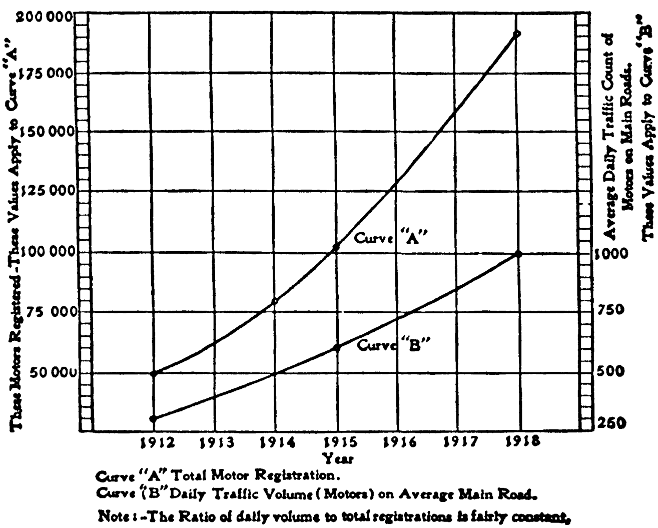


FIG. 5.—Comparison total motor vehicle registration and average rural road traffic volume. (State of Massachusetts.)

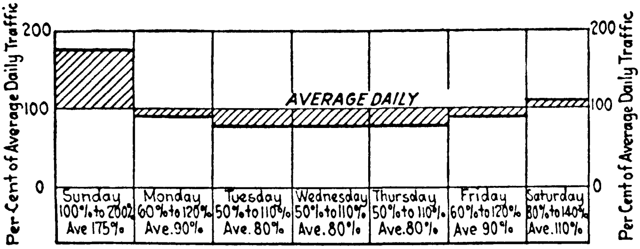
volume (see Fig. 5), although after a certain number of motors per capita is reached the volume of travel on rural roads is not greatly affected except on holidays. During the past 4 years the registration in New York State shows a large increase, but our traffic counts show only a small average increase, particularly on local roads.

Average daily traffic volume on rural roads is not so excessive as commonly believed by the city dweller, who bases his opinion on holiday volume, nor is it likely to continue to increase as rapidly as much of the automobile propaganda would make us think. The growth of motor vehicle traffic on rural highways has been rapid in the past few years, but it cannot be expected to continue at the same rate much longer, for a reasonable limit of the number of cars

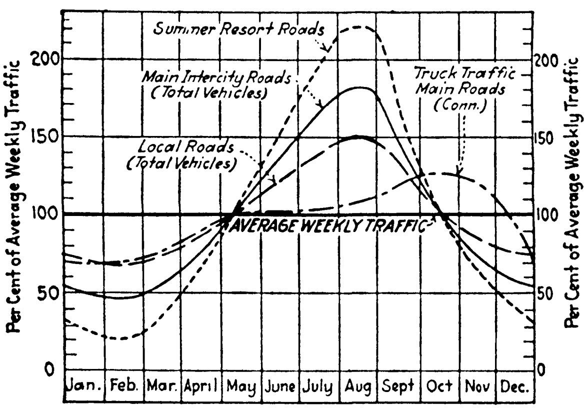
Hourly Fluctuation.—Hourly maximum 150 to 450% of hour average for a full day (24 hr.). Average 300%.

Ratio of Night to Day Traffic (summer months).

Night traffic (7 p.m. to 7 a.m.) ranges from 20 to 60% and averages 35% of day traffic (7 a.m. to 7 p.m.).



Typical daily fluctuation from weekly average (summer months).



Typical weekly fluctuation from yearly average (western N. Y.).

Based on personal records of W. G. Harger supplemented by traffic counts of Connecticut, Maryland, and Pennsylvania (see U. S. Bureau of Roads *Reports* for more complete data).

Diagram B.—Traffic fluctuation.

NOTE.—Individual roads may vary considerably from these curves (agricultural roads generally carry their peak loads in October, etc.), but these data provide some basis for coordinating short-time counts with yearly totals. Considering ratios of night to day traffic and seasonal variations, the author has been in the habit of using the following rough rules for deriving total yearly volumes in Western New York from 12-hr. daylight counts (7 a.m. to 7 p.m.) on Friday and Saturday in August.

Summer-resort roads.......	365 days × 70% of 12-hr. count
Main intercity roads.......	365 days × 80% of 12-hr. count
Local roads...............	365 days × 90% of 12-hr. count

FIG. 6.—Typical traffic fluctuation.

which are likely to be operated for any extended mileage on rural highways is being rapidly approached; normal growth can be expected, but at a greatly reduced rate. At the present time not over 2% of the total rural road mileage of New York State carries an average of over 1500 vehicles per day (year-round average), and it is not likely that it is necessary to figure on more than 3 to 5% being subjected to a higher volume than 1500 daily average during this generation.

This indicates that extreme pavement width or the extensive use of very high-grade pavements on a large mileage of road have not much foundation in economic necessity. The effect of a conclusion

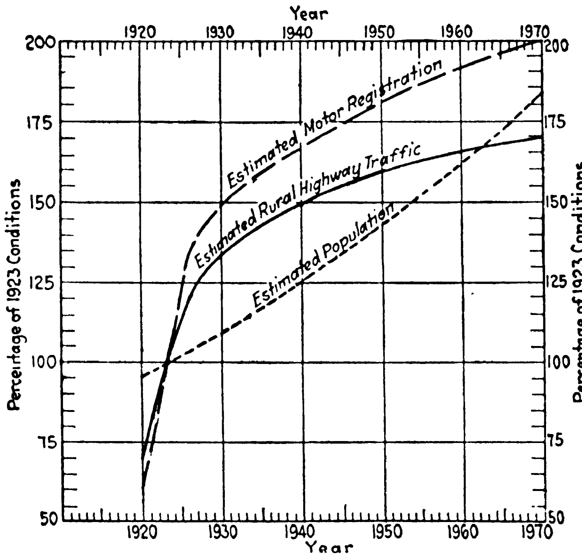


FIG. 7.—Estimated probable average increase in traffic volume. Division No. 4 Western New York.

of this nature is discussed in more detail under Classification (pp. 46 to 48).

Detail Data on Traffic Volume.—Present and future volume of traffic controls engineering design of widths, strength, alignment, and grades and gives the basic data for economic tests of design. For a large mileage of state system roads quite accurate general average laws can be established by careful traffic census counts taken from year to year, which are sufficiently reliable for broad conclusions. Individual roads, however, will vary greatly from the average and any estimates of future volume for detail design purposes must depend on a detailed study of prevailing and probable future volume, depending on adjacent territorial development modified by the expected improvement of adjacent roads. If this material is carefully analyzed, however, even individual roads can be given a broad general classification which helps materially with a rational and economic design. Examples of such estimates are given on page 33.

The reader is referred to the U. S. Bureau of Public Roads *Bulletins* for the best traffic census data in different parts of the country. In case these data cannot be easily obtained, the following data illustrate typical traffic fluctuation for conditions similar to the northeastern states.

Figure 6 illustrates typical traffic fluctuation (page 30.)

Figure 7 gives the general basis for estimating future traffic in western New York, and the details of its application to specific roads is explained as follows:

ESTIMATING FUTURE TRAFFIC VOLUME (FIG. 7) FOR ECONOMIC ANALYSIS

(From a paper prepared by W. G. Harger for the Michigan A.A.E., February, 1924)

"The kind and volume of future traffic are the fundamental factors of economic analyses, which are only possible where maximum loading and speed of trucks are rigidly controlled. Traffic volume controls the selection of reasonable types of pavement considering first cost, maintenance and renewal; it controls rational limits of expenditure for relocations and grade improvements and sets the value of the improvement from the standpoint of reduction in vehicle operating costs.

"For the original construction design of a new system of modern highways, it is impossible to forecast with exactness probable future traffic volume, but a fair approximation can be derived from a study of similar districts, where the road systems have been completed. As previously stated, economic analyses of highway design become of increasing practical value for reconstruction programs financed by direct vehicle taxation, and for such programs where the system has been completed and traffic routes established very close estimates of present traffic can be obtained by means of careful traffic counts similar to the Connecticut Census now under way, under the direction of the U. S. Bureau of Roads. Future traffic is largely a matter of judgment, but there is every reason to believe that we have enough data to make reasonable forecasts for the purpose of arriving at rational general conclusions. In making such forecasts more liberality is justified in connection with economic analyses of relocations and grade reductions than for pavement construction, as pavements are temporary at best. That is, an allowance for 50 years' growth is perhaps reasonable in connection with relocations grading and bridges and for 15 years' growth in connection with pavement design. The percentage of increase for these periods will, of course, vary for each road, depending largely on how near each locality has reached the saturation point of ton miles or car miles traveled on rural highways, and it is also affected by additional mileage of improved roads to be constructed during these periods which tend to reduce congestion.

"For western New York conditions (one motor per five and one-half people in 1923) the author has been in the habit of estimating average future volume on the basis of 170 % of present traffic (1923) for grading estimates and 150 % for pavement estimates which are modified for each road by the probable effect of the improvement of adjacent highways (see Fig. 7 and examples given beyond). These percentages will be reduced from year to year as the district more nearly reaches the saturation point. The comparatively small difference in the allowance of 50- and 15-year periods is due to the fact that this district is approaching the saturation point, for present population, except on the main roads carrying foreign traffic on which large future increases will be prevented by the construction of parallel routes. Our rural population is not increasing and the ratio of cars per capita for farm population has about reached the saturation point. The increase in population for this district will be due largely to the increase in the size of the city of Rochester. After 1930 the registration curve (Fig. 7) should not show much increase over the population curve. Up to 1930 there will be considerable increase in the registration curve, but our traffic counts for the last 6 years show that the volume of traffic on rural highways does not keep pace with registration curve.

Examples of Figuring Future Volume

"The relation between short-time counts and average yearly volumes is shown in Fig. 6, based on the Maryland and Connecticut data. This gives an approximate basis for judgment in deriving yearly volumes from short-time counts."

Example 1. *Clover Street Road* 294, *Monroe County.*—"1923 Census (12-hr. count in August) shows 550 vehicles (average mixed traffic). The year-round average daily 24-hr. volume is probably about 90 % of the 12-hr. count. This road will not be affected by additional adjacent improvements, as the system is complete, so an average daily volume of 500 × 170 % = 850 vehicles for grading estimates or 500 × 150 % = 750 vehicles for pavement estimates is estimated. This estimate places this road under Class II traffic.

Example 2. *Pittsford Palmyra Road* 766, *Grade-crossing Elimination Near Pittsford Village.*—"The 1923 Census shows 750 vehicles daily, average for present condition of the system. Within 3 years the Pittsford Victor Road will be completed, which will divert at least 60 % of traffic from Road 493 to Road 766. The 1923 Census on Road 493 shows 1900 daily. No other changes in the system are liable to affect traffic on Road 766. Future volume is, therefore, figured in two parts.

Estimate for Relocations

Normal growth..................750 × 170 % =	1275 daily
Diversion growth.....(60 % of 1900) × 170 % =	1830 daily
Estimated future total........................	3105 daily
Say.	2500 to 3500 daily

"This estimate places this improvement on Class I traffic basis."

Example 3. *Leroy Caledonia Road* 5463, *State Route* 6, *Genesee County.*—"The 1923 Census shows 3400 average daily. Within 2 years the completion of the Rochester-Batavia Road, via Churchville, will divert considerable traffic, and within 10 years the completion of the Avon-Buffalo Road via Alexander, will cause a future diversion of traffic.

"If there were no future diversions, reconstruction design would have to be considered on the basis of 3400 × 170 % = 5800 vehicles daily, which would require a 27′ width of high-type pavement. Considering the diversions, the future traffic will probably not exceed 2500 to 3500 daily average, which can be amply served with either an 18 or 20′ width of rigid pavement; that is, this road remains Class I, but it is not likely to become Class I plus.

"These examples illustrate the value of future estimates for general conclusions and their limitations for hair-splitting analyses of pavement design."

The effect of traffic volume on widths, pavement types, etc., is taken up in detail under Economic Engineering Design (p. 55).

7. Traffic Safety and Convenience.—Causes of accident are shown in Tables 9, 10, 11, and 12. It can be seen that at least 50% of accidents are caused by reckless or careless driving or defects in vehicle equipment, and are not preventable by highway safeguards.

(text continued on page 36.)

TABLE 9.—CAUSES AND LOCATION OF ACCIDENTS
1922 and 1923 accidents in Wisconsin (State Road System)

	1922 and 1923	Per cent of total
Total number of accidents	2,981	
Number of people involved	10,258	
Number of people killed	267	
Number of people seriously injured	794	
Number of people slightly injured	2,506	
Accidents occurring on straight road	2,044	68.6
Accidents occurring at curves and corners	479	16.0
Accidents occurring at railroad crossings	179	6.0
Accidents occurring at crossroads	138	4.7
Accidents occurring on hills	34	1.2
Accidents occurring at miscellaneous places	107	3.6
Accidents due to reckless driving	1,628	54.9
Accidents due to improper or no lights	218	7.2
Accidents due to broken car mechanism	151	5.0
Accidents due to intoxicated drivers	154	5.1
Accidents due to weather conditions	167	5.7
Accidents due to car on wrong side of road	74	2.5
Accidents due to narrow bridges and culverts	53	1.7
Accidents due to miscellaneous causes	536	18.0

TABLE 10.—CAUSE ANALYSIS, STATE OF MASSACHUSETTS
(Fiscal year 1923)

Principal Classes of Circumstances at Time of Accident	Fatal Automobile Accidents
Too fast for conditions	218
Inattention	122
Child darting in front of moving vehicle	75
Pedestrian running across street	47
Intoxicated operator, or had been drinking[1]	46
Pedestrian from front or behind vehicle	33
Wrong side of road	37
Confused operator	32
Defective brakes	30
Inexperience	31
Too close to other vehicles	24
Obstructed view	20

[1] In these 46 liquor cases, 62 persons were killed and 53 injured.

Accidents on Indiana State Highways

Collisions

	Mar. 1, 1922–Sept. 30, 1922	Oct. 1, 1923–Sept. 30, 1924	Oct. 1, 1923–Sept. 30, 1924	Oct. 1, 1924–Sept. 30, 1925
Total number accidents reported	157	332	370	654
Collision with other machines	49	109	160	302
Collision with railroad trains (steam)	4	16	6	19
Collision with guard rails	5	14	15	17
Collision with bicycles	1	0	0	1
Collision with culvert headwalls	7	9	11	14
Collision with telephone poles	7	21	12	33
Collision with wagons	2	11	6	7
Collision with electric cars	1	6	5	11
Collision with fences	8	12	12	21
Collision with animals	2	7	8	5
Collision with bridges	5	20	30	40
Collision with miscellaneous objects	20	48	14	22
Causes of accidents				
Fast driving	99	140	181	335
Steep grade	8	14	21	28
Sharp curve	41	51	33	78
Lack of guard rail	8	26	21	22
Wet surface	3	13	16	28
Deep ditch	5	15	13	27
Intoxication	23	25	40	66
Glaring headlights	9	17	25	54
Narrow roadway	29	59	41	45
Obstructed view	9	29	24	47
Miscellaneous causes	25	37	14	37
Deaths and injuries from these accidents were reported as follows:				
Accidents reported	157	332	370	654
Deaths	30	38	50	103
Injuries	133	196	255	529

Table 11.—State of Connecticut, Jan. 1–Aug. 15, 1924

Cause	Number	Per cent distribution
Recklessness of motorist	5,929	57.8
Carelessness of child pedestrian	382	3.7
Carelessness of adult pedestrian	777	7.6
Carelessness of contributor	1,083	10.6
Equipment	537	5.2
Miscellaneous	180	1.7
Minor accidents	1,376	13.4
Total	10,264	100.0

TABLE 12.—FATAL-ACCIDENT SUMMARIES FROM NATIONAL AUTOMOBILE CHAMBER OF COMMERCE

Totals reported for seven months, 1924	
Total motor fatalities reported	1390
Collisions between motor vehicles	248
Motor vehicles *vs.* electric railway	126
Motor vehicle at railroad crossing	91
Motor vehicle overturning	57
Motor vehicle striking stationary object	44
Causes of Fatalities	
A. Where fault of motorist was a major factor:	
Speeding	211
Violating rules of road	141
Inattention	35
Inexperience	51
Confusion	36
Intoxication	62
Total	536
B. Where physical conditions were major factors:	
Fog, snow, or rain	86
Skidding	73
Defect in vehicle	64
Too strong lights on vehicle	20
Poor street lighting	8
Defect in road	27
Confusion in dimming	13
Total	291
C. Where fault of pedestrian was a major factor:	
Adult jaywalking[1]	375
Children coasting in street	24
Pedestrians confused	71
Children crossing in middle or playing in streets[1]	224
Intoxication	13
Physical disability	20
Children stealing rides	00
Total	727

[1] The National Automobile Chamber of Commerce notes that, while these cases are technically the fault of pedestrians, yet the motorist must be ever alert to unexpected traffic violations of those on foot. Safety, alertness, and courtesy, rather than insistence on right of way, should be the motorist's rule.

Preventable accidents can be reduced by means of warning of dangerous physical conditions which have not yet been made safe and by means of safe highway construction, considering ample width, easy curves, easy grades, non-skid pavement surfaces, clear view, railroad-crossing protection, night illumination, and the segregation of pedestrian and motor traffic.

The amount of money that it is desirable to spend on safeguards is largely controlled by the volume of traffic. Extreme danger should be avoided on any improved road, but on the lighter-traveled roads considerable must be left to the care of the driver. On such roads about all that is justified are danger signs and cheap guard rails that warn instead of actually protecting, and on many mountain roads even cheap guard rail is out of the question. On heavy-travel special-service roads, all possible safeguards should be employed, such as the elimination of railroad grade crossings, substantial strong concrete guard rail or retaining walls, widening and banking of pavement on curves, pavement center-line traffic marking, a safe "sight distance" ahead at all times, hard well-kept shoulders, shallow ditches, warning and guide signs for the direction of travel, sidewalks for pedestrians, and night illumination. Safety policy is summarized as follows: On light-traffic roads confine safety provisions to warnings. On heavy-traffic routes spend all the money that is necessary to make the road as nearly foolproof as possible.

The details and cost of these safety devices are discussed under the detail design data for the various items involved.

Speed of Traffic.—Permissible and desirable speed of traffic is considered from the standpoints of safety, driver's preference, road-carrying capacity, and time of travel on through routes between populous termini. Permissible speed is a matter of dispute between even the best authorities in highway work, but there are certain well-established fundamentals. Safe speeds depend on type, weight, and condition of the road vehicle, expertness of vehicle operator, highway alignment, grade, pavement width, type of surface, and spacing of vehicles (congestion), and will vary from 10 to 40 m.p.h. From the standpoint of safety alone it is well established that, for any given existing highway condition, increased speed increases danger. At points where a low speed is required for safety, highway improvements can generally be made which permit reasonable increase in speed with practically no increase in danger, but such improvements are often very expensive and it is necessary to arrive at reasonable limits of speed which will be used as a basis for design.

It is self-evident that road users desire high speed and are exhilarated by "stepping on the gas," but it is also evident that this tendency is the cause of most accidents; that long, straight, smooth, and wide highways encourage high speeds, which are productive of a large number of accidents under what appear to be safe driving conditions, and that it is hardly justifiable to spend large sums of money to encourage excessive speed. From the standpoint of reasonable safety and reasonable desire for speed, designs which are safe for 25 m.p.h. on curves in rough country and 35 m.p.h. for average driving conditions in ordinary topography should serve satisfactorily for main roads subject to mixed traffic. These limits can be reduced for the less used roads.

Maximum road-carrying capacity depends on speed and the necessary spacing between vehicles moving at different rates of speed. From the standpoint of road-carrying capacity recent studies (1925) have shown that the greatest number of cars can be carried past a set point when the average speed of travel is about 22 m.p.h. and that the tonnage capacity of the road is reduced by either increasing or decreasing this speed limit. The accompanying table shows the effect of speed on road capacity. Data of this kind are, of course, only useful for police regulation of congested roads.[1]

Speed of travel, m.p.h.	Capacity of road in percentage of maximum
15	90
20	100
25	100
30	95
35	90
40	80

From the standpoint of reduction in time of travel between cities, some Engineers now advocate designs permitting 50-mile speeds, but such speed requires so-called super highway design, which eliminates grade crossing of intersecting roads, requires extremely high standards of alignment grade and pavement width, and limits the road solely to the use of high-powered cars making the through run. There are very few conditions where such special highways are necessary or economically justified at the present time (1926).

The following quotation from James E. Kelly discusses safe city and rural speeds:

"I believe that an average speed of more than 15 m.p.h. is unsafe where blocks are 0.1 mile or shorter and intersections obscure. Where blocks are 0.3 mile or more long this can be increased to 20 miles with safety if there is very little parking. Parked cars along streets, obscure driveways, and pedestrians, particularly children playing on streets, are the greatest menace to safety and hence retard traffic.

"In the open country with long straight stretches of highway, well-banked curves of long radii and clear view, 40 to 45 m.p.h. is safe for an experienced driver with a well-built carefully maintained car. A speed greater than 45 m.p.h. generally brings grief to someone, usually the driver."

8. Traffic Range.—Available data indicate that probably 70 to 95% of road traffic can be classed as local service; that is, it has its origin and finish within a comparatively short distance, say 30 to 40 miles, and consists of hauling garden truck to cities or produce to shipping points, ordinary business intercourse, and short pleasure trips. The other 5 to 30% may be classed as long-distance traffic, consisting of pleasure touring, commercial travelers' cars, and trucking between cities. These ratios of general traffic do not, of course, apply to any one road and may be actually reversed on certain special touring roads, but they probably apply to road systems as a whole for most counties and states and indicate the general administrative principle that local traffic is entitled to first consideration in the location of roads and their design, except for a comparatively small mileage of special-service highways.

[1] For normal capacity two lane highways see page 28.

The statistics compiled by the National Automobile Chamber of Commerce for 1920 show that 30% of all automobiles and trucks are owned by farmers; that 60% of all motors are registered from towns of 5000 inhabitants or less and the rural districts. This is a very conservative basis for deriving the minimum volume of traffic originating on or using side roads. The general city public and even engineers are inclined to think too much of the main roads in connection with any state or national program of highway improvement. This is natural, as the main roads are more spectacular; they probaby carry 90% of the traffic during some part of the journey. A system of improved main roads gives quite complete service to intercity travel; it gives quite complete service to tourist travel, but it gives only partial service to local travel, and local traffic is a very large proportion of the total travel on our roads. A large percentage of traffic either originates on or uses a side road during some part of its journey; that is, the main roads only provide direct contact service to about 10 to 20% of the producing area of the district, and this element of direct contact is a fundamental service of roads. It is, therefore, extremely poor policy to over-emphasize the importance of the main roads to the extent of disregarding a reasonable treatment of the secondary roads. Main roads are entitled to first consideration and better construction than the local roads, but they are not entitled to needlessly expensive construction which tends to delay too long a reasonable treatment of the system as a whole. The general character of the system must progress as rapidly as possible up to the standard of general utility; beyond that standard there is no harm in concentrating exceptional expenditures on special roads. The difference in volume of traffic on the local and main roads calls for a wide variation in suitable pavement design. The administrative problem considers the general suitability of pavement types under different classes and volume of traffic.

TRIP MILEAGE OF PASSENGER CARS USED FOR BUSINESS AND NON-BUSINESS PURPOSES[1]

Trip mileage	All passenger cars, %	Business cars, %	Non-business cars, %
0–19	55.7	65.3	48.0
20–39	16.3	14.4	17.9
40–59	9.2	8.2	10.1
60–79	5.0	3.9	5.9
80–99	3.6	1.7	5.1
100–149	4.3	3.3	5.1
150–199	1.7	1.2	2.1
200–299	2.7	1.5	3.7
300–399	0.5	0.2	0.7
400–499	0.2	0.1	0.4
500 and over	0.8	0.2	1.0
Total	100.0	100.0	100.0

[1] Connecticut Traffic Census, Public Roads, August, 1926.

PERCENTAGES OF ALL PASSENGER CARS USED FOR BUSINESS AND NON-BUSINESS PURPOSES, CLASSIFIED ACCORDING TO TRIP MILEAGE

Trip mileage	Business cars, %	Non-business cars, %
0–19	52.2	47.8
20–39	39.3	60.7
40–59	39.4	60.6
60–79	34.8	65.2
80–99	21.6	78.4
100–149	34.5	65.5
150–199	30.9	69.1
200–299	24.6	75.4
300–399	17.6	82.4
400–499	17.2	82.8
500 and over	12.6	87.4
Total	44.5	55.5

DISTRIBUTION OF NET TONNAGE OF COMMODITIES TRANSPORTED BY MOTOR TRUCK OVER THE CONNECTICUT STATE HIGHWAY SYSTEM BY LENGTH OF HAUL

Length of haul, miles	Proportion of total net tonnage, %	Length of haul, miles	Proportion of total net tonnage, %
0–9	36.3	70–79	2.6
10–19	19.2	80–89	0.8
20–29	11.6	90–99	1.0
30–39	9.1	100 and over	8.6
40–49	4.0		
50–59	4.6	Total	100.0
60–69	2.2		

9. General Suitability of Types.—It is not necessary to go into the details of the minor advantages of various standard pavements; these will be discussed under the general principles of Engineering Selection (p. 58). Administrative action, however, properly considers the fundamental desirability of general types, their relative cost (both initial and final), and their natural rate of depreciation as far as they affect methods of taxation, terms of bond issues, and maximum tax rates.

Two general classes of pavement are considered suitable for modern highways: rigid and flexible. The distinctive feature of the rigid type is a non-elastic concrete base which distributes the concentrated traffic wheel loads over a safe area of the natural subsoil by slab action. All sorts of surfacing are used (sheet asphalt, brick, stone block, asphalt block, or exceptionally strong concrete). The distinctive feature of the flexible type is a base course constructed of some form of macadam, gravel, field stone, or Telford, distributing the wheel loads over a safe area of the natural subsoil by means of the depth of the course and the mechanical interlocking or friction of the integral fragments of the material. This type has all sorts of surfacings, the most common surface being some form of modern macadam, but under certain conditions it is proper to use sheet asphalt or similar surfacings and almost any type of block, provided the joint filler is flexible to permit siight settlement without rupture. Rigid pavements are desirable for heavy traffic. The flexible pavements are desirable for moderate and light traffic.

The advantages and disadvantages are as follows: Rigid pavements are destroyed by the action of the elements as well as by traffic. They crack due to the settlement of new fills or frost heave; they shatter due to changes in temperature; they disintegrate due to vibratory fatigue and frost action. They are comparatively difficult to repair and prohibitive in cost except for rich communities. They, however, handle heavy auto trucking more satisfactorily than macadam construction. They need comparatively little surface maintenance for the first few years, and for this reason traffic is inconvenienced less than on macadams; they last a longer period without reconstruction than macadam pavements, and traffic is therefore interrupted less; they bridge over small areas of weakness in the subgrade, such as culvert backfills, etc., better than macadams. There is no question but that they are desirable on roads carrying a large amount of heavy-truck traffic.

For moderate traffic requirements, however, the flexible type of pavement is probably more suitable. It complies better with the usual conditions. It generally costs less to construct; is not seriously damaged by settlement of new grading or frost heave; can be easily and cheaply repaired; can be strengthened gradually by the addition of stone to meet practically any loading; and when a solid foundation has been accomplished can be recapped with a higher-grade surface, which rids it of the continuous-maintenance draw back.

It is well to bear in mind that the quality of rigidity is not inherently desirable, considering the fact that the earth foundation is always susceptible to heave and settlement and that this will occur to some extent no matter what precautions are taken. Rigid design should be resorted to only when the desired result can be accomplished more cheaply by this means than by the flexible form of construction; this occurs under exceptionally heavy-unit traffic. The fact that rigid pavements, in general, include a higher type of surfacing than macadam constructions leaves the impression in the popular mind that such a surface is a result of rigidity, while, as a matter of fact, there is sufficient data to state with reasonable assurance that, unless the pavement is subjected to an unusually large volume of heavy-unit motor trucks, a first-class macadam, field stone, or Telford base is often superior to concrete base for asphalt or small block surfacings on account of the reduction in frequency of temperature cracks.

The present rather extreme tendency favoring rigid construction under moderate traffic is based largely on the record of failures of

(text continued on page 46.)

TOTAL MILES OF ROADS IN U. S. AT END OF 1924—(STATE, COUNTY AND LOCAL ROADS COMBINED)
Compiled from Bureau of Public Roads Statistics and State Reports, by Portland Cement Association

States	Grand total mileage	Unimproved and earth partially graded	Earth to established grade and drained	Total miles of road surfaced	Low type		Medium type	
					Sand-clay	Gravel, etc. untreated	Water-bound macadam untreated	Surface treated macadam and gravel
Alabama	61,491.2	48,833.4	157.8	12,500.0	7,075.1	4,186.0	698.9	240.2
Arizona	22,355.3	17,055.7	2,238.3	3,061.3	648.3	1,906.6		
Arkansas	74,865.0	66,220.5	2,806.0	5,838.5	150.2	4,499.9	345.2	76.4
California	75,618.0	30,117.8	30,238.0	15,262.2	....	6,873.0	1,005.4	566.0
Colorado	67,620.5	25,867.3	33,003.3	8,749.9	...	8,550.0		
Connecticut	12,219.6	7,277.9	2,530.0	2,411.7		83.5	111.5	1,501.8
Delaware	4,410.8	1,923.5	1,790.0	697.3	. .	50.0	30.0	153.5
Florida	28,413.5	19,369.9	44.7	8,998.9	3,171.5	1,505.6	679.4	1,291.8
Georgia	98,364.6	52,978.6	26,198.8	19,187.2	14,128.4	3,553.8	219.6	140.7
Idaho	34,816.3	11,217.9	12,630.0	10,968.4	68.2	10,342.5	325.0	
Illinois	96,328.1	80,437.4	1,717.8	14,172.9	.....	6,783.0	2,623.1	138.0
Indiana	80,533.7	40,514.3	130.7	39,888.7	..	25,461.1	12,028.6	238.0
Iowa	104,624.2	97,134.7	2,762.0	4,727.5		4,223.2		
Kansas	129,125.0	127,395.4	319.8	1,409.8	65.0	430.1	240.0	63.0
Kentucky	68,704.0	50,950.0	687.4	17,065.7	70.0	3,698.1	11,125.5	1,615.3
Louisiana	39,803.0	35,125.0		4,678.0		4,521.6	. . . .	87.1

Maine	20,939.8	16,966.6	8.9	3,964.3	8.2	2,756.8	605.4	347.6
Maryland	14,151.8	8,722.0	1,807.0	4,357.2	414.0	1,168.1	189.0	1,419.2
Massachusetts	19,098.4	11,775.4	12.8	7,310.2	2.7	3,126.5	96.2	2,019.3
Michigan	77,284.1	54,751.1	1,534.8	20,998.2	78.0	16,208.0	1,254.0	1,035.4
Minnesota	107,827.0	81,650.4	3,866.4	22,310.2	3,067.3	17,523.7	83.4	67.4
Mississippi	58,880.4	48,647.9	991.6	9,240.9	366.8	8,193.3	100.2	69.4
Missouri	112,101.1	99,505.2	3,060.2	9,535.7	1,755.3	5,577.0	1,213.8	56.0
Montana	64,176.9	62,966.6	280.1	930.2	......	889.7	......	0.6
Nebraska	87,391.8	82,999.7	3,251.2	1,140.9	386.5	676.4	0.7	
Nevada	22,778.8	20,178.6	1,695.2	905.0	......	823.6	0.5	10.2
New Hampshire	13,089.0	11,852.2	10.4	1,226.4	......	322.9	9.7	728.4
New Jersey	17,716.2	74.9	10,193.2	7,448.1	......	2,838.5	1,342.4	739.1
New Mexico	47,030.7	43,939.1	680.7	2,410.9	731.0	1,615.8		
New York	81,873.0	56,886.3	447.4	24,539.3	......	5,229.8	3,484.4	6,036.5
North Carolina	67,906.7	48,867.4	889.6	18,149.7	12,515.7	3,291.9	400.3	119.8
North Dakota	106,498.4	103,899.5	1,544.0	1,054.9	......	1,050.1		
Ohio	84,532.1	50,667.5	283.8	33,580.8	501.0	13,748.5	13,113.8	951.8
Oklahoma	134,262.0	128,975.3	3,618.7	1,668.0	......	885.5	29.7	267.8
Oregon	49,770.0	32,264.0	7,386.0	10,120.0	......	8,786.0		
Pennsylvania	92,020.5	72,007.7	3,913.6	16,099.2	0.5	665.3	696.2	2,746.8
Rhode Island	2,320.6	1,491.1	178.0	651.5	......	60.5	47.8	198.0
South Carolina	64,408.5	48,038.7	7,288.9	9,080.9	8,126.9	588.7	33.9	18.5
South Dakota	115,805.4	109,803.7	3,987.7	2,014.0	......	2,012.8		
Tennessee	64,726.7	41,538.1	10,855.6	12,333.0	......	5,597.4	4,865.8	830.8
Texas	167,685.0	145,791.1	2,845.0	19,048.9	2,109.1	13,958.3	571.2	1,419.2
Utah	23,381.1	18,437.9	1,989.6	2,953.6	697.6	1,920.6	12.0	3.6
Vermont	14,861.0	9,825.6	554.0	4,481.4	......	4,341.8	9.0	75.8
Virginia	62,106.4	51,563.0	2,202.2	8,341.2	2,528.8	1,744.2	762.4	1,487.2
Washington	48,883.7	22,799.3	9,559.2	16,525.2	......	14,205.9		
West Virginia	35,565.8	32,993.0	937.7	1,599.1	......	173.4	231.1	47.7
Wisconsin	78,964.2	20,456.3	34,070.7	24,437.2	4,115.8	16,915.9	1,389.7	
Wyoming	46,319.2	45,071.8	523.2	724.2	......	683.8	......	2.2
Totals	3,003,649.1	2,297,827.2	237,758.0	468,798.3	63,681.9	244,248.7	59,974.8	26,810.1

TOTAL MILES OF ROADS IN U. S. AT END OF 1924—(STATE, COUNTY AND LOCAL ROADS COMBINED)—*Continued*
Compiled from Bureau of Public Roads Statistics and State Reports by Portland Cement Association

States	Medium type		High type						
	Bituminous macadam by penetration	Misc.	Sheet asphalt	Bituminous concrete	Cement concrete	Block pavements			
						Brick	Asphalt	Wood	Stone
Alabama	152.4		24.9	104.8	17.5			0.2	
Arizona			15.0	91.4	400.0				
Arkansas	246.7		41.4	296.9	181.8				
California	1,370.6	900.0		1,087.9	3,459.3				
Colorado					199.9				
Connecticut	280.8		4.0	143.9	284.5	1.7			
Delaware	39.6			20.5	396.8	6.9			
Florida	334.2		785.2	64.7	257.3	764.2	145.0		
Georgia	347.7		55.3	82.2	658.9	0.6			
Idaho		70.0	5.4	116.2	41.1				
Illinois	67.8		33.0	56.0	4,232.0	232.0		8.0	
Indiana	396.3			172.0	1,370.9	221.3		0.5	
Iowa					450.4	53.9			
Kansas	72.9				426.5	112.3			
Kentucky	228.1	97.0	56.0	13.8	147.7	9.2		5.0	
Louisiana	23.4			18.0	9.9	18.0			

Maine	183.6	....	3.8	1.5	57.4				
Maryland	70.0	189.0	56.0	28.2	816.0	7.7			
Massachusetts	1,557.9	....	....	318.2	186.7	0.8	....	0.1	1.8
Michigan	201.0	231.9	2.0	280.0	1,694.1	13.8			
Minnesota	10.6	....	8.5	88.1	535.5	13.2	....	12.5	
Mississippi	232.9	....	6.7	19.8	232.6	19.2			
Missouri	159.9	....	1.0	5.6	757.1	10.0			
Montana	5.5	....	....	2.3	32.1				
Nebraska	....	....	2.6	10.2	45.0	19.5			
Nevada	24.0	....	....	1.6	45.1				
New Hampshire	98.2	....	....	59.2	8.0				
New Jersey	725.8	....	291.4	706.8	664.2	40.9	3.1	6.3	89.6
New Mexico	....	....	....	0.7	63.4				
New York	6,489.2	....	16.5	215.9	2,709.8	329.5	23.4	0.6	3.7
North Carolina	186.6	0.5	12.0	802.3	754.0	66.1			0.5
North Dakota	....	....	....	....	4.8				
Ohio	1,638.7	414.7	44.5	218.3	1,428.3	1,521.2			
Oklahoma	....	....	....	80.0	385.0	20.0			
Oregon	....	130.0	....	919.5	284.5				
Pennsylvania	349.4	7,796.3	207.3	603.3	2,471.6	548.3	6.1	3.4	4.7
Rhode Island	160.9	....	8.9	141.9	30.5	....	....	....	3.0
South Carolina	16.8		66.3	67.8	162.0				
South Dakota	....	....	....	....	1.2				
Tennessee	487.9	....	363.2	67.9	119.0	....	....	1.0	
Texas	357.5	....	106.4	112.0	372.8	42.4			
Utah	7.2	....	10.5	59.1	243.0				
Vermont	22.9	....	....	....	31.9				
Virginia	583.1	770.5	10.7	7.6	445.8	0.9			
Washington	151.6	160.0	21.8	266.3	1,657.3	62.3			
West Virginia	421.4	24.2	0.7	75.7	426.9	198.0			
Wisconsin	....	....	....	94.6	1,921.2				
Wyoming	....	....	....	27.1	11.1				
Totals	17,703.1	10,784.1	2,261.0	7,549.8	31,132.4	4,333.9	117.6	37.6	103.3

old, thin, inadequately designed and maintained macadam roads. Very few of the more recent, carefully designed macadams have been unsatisfactory under a traffic of less than 2000 vehicles daily. A small percentage of all types of pavement fails either on account of poor design, poor construction, or poor maintenance, but there is no such thing as type failure if moderately good judgment is used in the design and maintenance. Road failures are personal human failures.

Considering that road improvements are generally handicapped by a shortage of funds, that, of necessity, they must advance by successive stages, that the flexible types are usually the cheapest in first cost, that they are also economical in the long run for a moderate volume of traffic, and that the surface maintenance problem can be minimized by using various types of surface, it is generally good policy to give these types the preference for the first stages of improvement programs; even for the final stages they are the best investment for the community for probably 90 to 95% of the total road mileage in most districts.

To illustrate this more definitely, the following extract is quoted from the report on the Monroe County system (New York State):

"We advocate the original construction of crossroads of thick modern water-bound macadam utilizing local materials as much as possible and maintained by surface oiling.

"We advocate the original construction of our secondary radial roads, of penetration, bituminous macadam, utilizing local materials to their fullest reasonable extent and maintained by surface oiling.

"We advocate the construction of our main trunk-line heavy-hauling roads of rigid pavements, using the best materials that can be obtained, but varying the type to secure, in each case, the cheapest first-cost pavement, always considering the possible use of local materials proper for the type of road in question. For these roads we have no choice between cement concrete, brick, sheet asphalt, asphalt block, or stone block on concrete bases.

"We advocate the gradual resurfacing of the heavier-traffic macadam roads with Topeka mix, small brick cubes, etc. We have successfully utilized this method in reducing high surface-maintenance costs where the macadam foundation was solid enough for the traffic, and have adopted this method for a number of our roads. We have examples which have stood a 10-years' test successfully.

"We believe that the community has been better served by constructing 10 miles of macadam in place of a possible 6 miles of rigid pavement.

"We believe that the county has been better served in the past and will be best served in the future by variable road designs using for the majority of the mileage modern macadam for the original construction, later modified, if necessary, for a very limited mileage by recapping with a lower-maintenance-cost surface. We advocate rigid pavements eventually for approximately 10% of the total mileage of our roads and for approximately 35% of our state system."

10. Classification of Roads.—In order to strengthen the general discussion, it is, perhaps, just as well to indicate a little more definitely at this point the initial and final cost of roads under different classes of traffic and what types of pavement appear to be best suited for different volumes of travel.

Four general classes of highway can be considered:

Class I Roads.—These roads are usually located along natural transportation routes between large cities located at intervals of less than 100 miles. They may also occur as main radial roads out of cities of, say, 50,000 population and upwards for distances of

5 to 40 miles or as special industrial roads or village streets carrying the concentrated traffic of a large area. These roads generally constitute from 0 to 10% of the total road mileage of the district. They usually carry 2000 or more vehicles per day (10-hr. count in summer). They should have pavements designed to support the traffic of the heaviest commercial trucks permitted by law and should have a pavement width of at least 18 and preferably 20'.

The types most suitable and economical in the long run, from the standpoint of even general utility, are any standard brick or bituminous concrete surface on cement concrete bases, or reinforced cement concrete pavements. Such roads cost today (1926) for the original construction, including grading, drainage, pavements, and incidentals, from $40,000 to $70,000 per mile. The yearly burden to the community, considering interest on first-cost investment plus yearly maintenance plus the yearly allowance for renewing pavement when worn out, amounts to 7 to 10% of the first cost of the road.[1] It seems reasonable for the community at large to assume the burden of original construction plus interest on first cost, and for vehicles to assume the burden of yearly maintenance and the renewal of worn-out pavement. The actual yearly maintenance and renewal charge for these roads are at first small and gradually increase, eventually reaching 3 to 5% of the first cost of the improvement and from there on stay practically constant. This is due to the fact that new pavements require a comparatively small yearly maintenance, which gradually increases until their renewal is necessary. The cost of renewal is high, and when any system of highways becomes old enough (15 to 25 years for this class of road), there will be a constant yearly renewal charge plus a constant yearly maintenance charge, which may easily amount to $2000 to $2500 per mile per year average (1926 scale of costs) for this class of road.

Class II Roads.—These roads are usually main automobile routes at greater distances from the cities than are Class I roads. They have a large touring-car traffic, medium-heavy farm traffic, and some heavy trucking. A Class II road generally carries from 800 to 2000 vehicles per day (10-hr. count in summer). They constitute approximately 0 to 20% of the total road mileage, depending on the general character of the district. They should be designed to handle such traffic, and require a pavement width of 15 to 18' supplemented by stone or gravel shoulders.

The type of pavement generally most economical for these roads, considering final cost and immediate traffic service, is thick modern bituminous macadam, or if the volume of traffic is close to the 2000 limit, it is possible that reinforced cement concrete may be desirable under advantageous conditions of material supply.

Such roads cost today (1926) from $25,000 to $40,000 per mile to construct and the total yearly burden amounts to about one-tenth of the original cost of construction.

The yearly maintenance and renewal charge for such roads will eventually (in 10 to 15 years) amount to approximately $1500 to

[1] See Chap. VII.

$2000 per mile (1926 scale of costs). For this class of roads this is properly a direct traffic charge.

Class III Roads.—These roads are usually secondary feeder or crossroads in well-settled districts or main roads in sparsely settled districts. They generally carry 300 to 800 vehicles daily (10-hr. count in summer). They constitute 5 to 30% of the total mileage. They may well be designed to handle at least a 2½-ton truck for all-the-year service and permit the use of 5-ton trucks in the dry season. They require a pavement width of 12 to 16′. The types of pavement generally most suitable for general utility are thick, modern, water-bound macadam with a surface application of oil, gravel oiled, or, if the travel approximates the 800 limit, the bituminous-macadam type can be considered.

These roads cost from $10,000 to $25,000 per mile to construct and the total yearly burden approximates one-tenth of the original cost. The yearly maintenance and renewal charge may amount eventually in 8 to 15 years to approximately $1000 to $1500 per mile per year (1926 scale of costs). For this class of road it is proper to charge this directly to traffic.

Class IV Roads.—These roads constitute the purely local service roads carrying a volume of traffic of less than 300 vehicles daily. They constitute 70 to 95% of the total road mileage. They may well be designed for the all-year-round use of 2½-ton trucks. They do not require much refinement in grading design or in the width of hard surface. A width of 8 to 12′ is sufficient.

The most suitable type is gravel or water-bound macadam. Earth or sand clay can be used as a temporary makeshift, but in northern climates they can only be considered as temporary expedients, as any type of surface not usable the year round for at least 2½-ton trucks is not a general-utility road.

Class IV roads cost $2500 to $10,000 per mile (1920) and the total yearly burden is about one-tenth of the original cost of construction. For this class of road it is probable that the community at large should assume the entire burden of construction, interest, maintenance, and renewal, which means eventually for the items of maintenance and renewal alone about $300 per mile per year (1926 scale of costs); this figure is reached gradually and becomes a fairly stable burden in 5 to 10 years after the roads are constructed.

11. Importance of Maintenance.—Effective maintenance is the best possible road investment the community can make. It increases the life and lowers the final cost of the improved roads. It decreases the cost of traffic operation and increases the comfort of travel. It speeds up the general effectiveness of a road system that is being gradually improved by construction or reconstruction. This last is the keynote policy for the poorer districts and is of vital importance to a large part of the country at present. For the poorer districts traffic service can be bettered at once, first, by systematic maintenance of all the existing roads; this does not solve the problem by any means, but it keeps it from getting worse; second, by the immediate construction of a large mileage of gravel or similar constructions on secondary roads suitable for moderate loads (up to 2½-ton truck and eliminating the large truck for year-round use) and giving these roads systematic maintenance. This solves the problem up to the standards of general-utility traffic for agricultural districts. It does not solve the heavy-traffic problem, but it helps and is better than waiting indefinitely for the completion of a system of boulevards. Third, the construction of thick modern macadam systematically maintained for the main roads and strong, expensive, rigid pavements systematically maintained for the main commercial hauling routes aggregating a comparatively small percentage of the total mileage. These roads meet heavy-travel requirements.

Fairly good immediate service for traffic is possible only with effective maintenance. It is both physically and financially impossible to solve the problem of improvement of traffic conditions over a large mileage solely by immediate reconstruction of the roads. Maintenance is not entirely eliminated no matter what type of road is built. Fairly efficient maintenance has been accomplished in many cases and is poor in others, but the fact that it can be achieved, provided enough attention is concentrated on it, means that it is possible to get somewhere with a serviceable road program in a reasonable time.

To illustrate the result of a lack of a temporary service program, New York State may be cited. This state has been improving its main roads by state and county aid for 20 to 25 years; the system is still incomplete in many cases. New York's construction program has been excellent; its maintenance of the roads after improvement has been fairly good for some years past, but very little has been done to better the condition of the existing roads at gaps in the improvement; that is, for the past 10 or 15 years, short gaps and detours in stretches of modern roads have caused needless inconvenience to the traveling public. Under the New York road program, the state does not assume the direct responsibility for the condition of the roads on the proposed system until they have been improved by state construction. As soon as a road is adopted as part of the system, the town in which it is located loses all interest in it and does nothing to keep it in condition, although it is supposed to keep it in shape until it is improved by the state. These gaps do not require large expenditures, but the conditions can at present be very materially bettered by comparatively small expenditures on temporary repair and maintenance. All travelers curse these gaps and are more than willing to have their license fee raised slightly to eliminate temporarily the worst features of such roads. It is probable that an extra charge between $1 and $2 per vehicle license per year applied to such work under direct state supervision would result in a tremendous improvement of the few gaps remaining in this particular state (1921).

12. Departmental Organization.—The success of a highway program really depends more on the individual character and professional ability of the men making up the organization than on any other feature; if the men are high grade, nothing radically wrong is likely to develop, and the designs are usually economical. If the men are inexperienced or second rate, no system of standardization or rigid red-tape procedure will prevent wasteful design and poor construction work. It is difficult to retain high-grade men in the public service, as usually the salaries are niggardly and the advancement slow and often not based on personal ability. One of the foremost state highway engineers resigned in 1920 from his position on the ground that it was impossible to develop an organization which could expend the state highway appropriations in a really efficient manner; that the trouble lay in low pay and the difficulty of advancing the best men; that, while they had succeeded in employing and training some very excellent talent, it was impossible to hold a reasonable proportion of the men and that the turnover was high.

This action expressed in a very effective way the quite universal handicap of a highway executive. Under the existing conditions, the solution most often adopted lies in retaining a few high-grade men as active subordinate assistants to politically appointed heads and rigidly standardizing detail procedure, utilizing quite low-pay men for the actual design and construction end of the program. Such a system produces a moderately good type of highway, as may be seen by observing the results to date, but any engineer actually acquainted with the working of this system will say that the cost of construction is needlessly high in most cases and that the community pays a tremendous amount of money every year because it has not sense enough to adopt a more liberal policy of salary and reward. High-grade engineering pays for itself in actual construction money saving 10 times over the additional salary cost, but this fact is not generally recognized by the public and, while road officials understand it, they either find it difficult to override public sentiment or actually prefer mediocre talent on account of the greater freedom which they have to carry out their own pet schemes. Until the entire organization from inspectors up to the chief is raised to a good standard of individual ability, waste is bound to occur.

Executives have said that they knew moderate-priced roads well designed, constructed, and maintained would serve satisfactorily, but that such results were hard to obtain under existing organization conditions and that in self-defense they considered it good policy to design a rigidly standardized $40,000 a mile highway even on comparatively unimportant Class III roads to get $20,000 worth of service. This expresses in an extreme way the continual waste that is more or less the result of niggardly public service employment policy.

Nothing takes the place of the individual judgment of the rank-and-file men of the organization. What most organizations need are high-grade detail designers and constructors who are encouraged to use their experience and judgment and who are not tied down too much by rigid standardization or their work crippled by demands for plans at such short notice that reasonable care and study cannot be given to each road. This statement must not be construed as an argument for unrelated work by a crowd of individualists, but departmental heads may well rid themselves of the idea that all wisdom originates at the official top and that standardization will take the place of brains.

Even with enough good men in the force it is necessary to have a definite organization plan which defines the duties and responsibility of each man. Too often the organizations are based on overlapping duties, indefinite responsibility, and conflicting authority, and it may be regretfully said that this is often purposely done to make it easier to "pass the buck." (See p. 1271 for definite statement of duties and responsibility of the construction engineering force.)

13. Contract Relations.—Assuming that the road is well designed, it is necessary to get it well built. Sound business relations between contractors and the directing engineering organizations is manifestly the only possible means of getting good work at a reasonable cost. Any element of unnecessary risk or uncertainty which the contractor must assume raises the bid price of the work. Any doubt as to whether the work will be let provided a reasonable bid is secured tends to keep away responsible contractors. Prompt decision and uniform treatment are essential. Reasonable profits are necessary to insure good work, for the community usually gets just what it pays for.

The author has heard public officials say that they figured to catch a sucker at every letting. They often did, but the result was that they either got a rotten job or had the difficulty of finishing the work themselves with all the usual complications. Fortunately, this attitude has few supporters to day and it may be stated as a general principle that uncertainty must be eliminated as far as possible, reasonable prices must be paid, and the size of contracts should be varied in order to interest organizations that can best handle the road in question.

The uniform use of long-mileage contracts is no more desirable than the use of short contracts. Large organizations have a high overhead and equipment charge and there are generally not enough of them to insure lively competition. They can afford to provide labor-saving machinery, which is a distinct advantage during times of labor shortage. The award of long contracts to large organizations is probably desirable for high-priced rigid pavements, particularly in sparsely settled communities.

Short contracts tend to encourage competition. They can generally be finished in one working season, which eliminates considerable uncertainty in the labor situation and the cost of materials. They can generally be handled with local labor, particularly in the well-settled districts. They cause less inconvenience to the traveling public during construction. They are probably desirable for the construction of roads in well-settled districts, particularly where the macadam form of pavement is used.

Uncertainty in bids can be reduced by complete and definite plans and specifications that have the reputation of being enforced; by definite statements of the requirements of materials and the location of acceptable supplies of these materials; by the publication of the engineer's estimate of cost, with a statement as to the maximum bid that will be considered in awarding the contract; and by the provision that, in case a responsible contractor makes the low bid under the limit stated and no award is made, he will receive a reasonable fee for making the bid.

To determine reasonable prices, every large state organization can afford to develop a construction department which can do certain jobs each year to gage reasonable construction costs and to take over for completion any contracts that may be canceled for non-performance.

14. Proportion and Economy in Design. *The Relative Importance of the Detail Elements of Design.*—Most road work can be classed as a step in progressive improvement; the highway is gradually bettered from a trail to a high-class, modern, heavy-traffic thoroughfare as its use or prospective use warrants the expenditure. In the majority of cases the money at hand is not sufficient for the complete construction of all the features that are desirable even at the time when the improvement is made, and it is never sufficient to build a road that will completely fill the requirements of the future. Some features have to be omitted or slighted. It, therefore, seems well worth while to encourage, first, the construction of reasonably good fundamental elements which act as a basis for the final improvement, and then, in logical order, as many of the other desirable parts as can be built.

It certainly pays to construct what is done so that it can be readily strengthened and widened as the future requires, without losing the benefit of the previous work. The following tentative list illustrates an order of importance of design elements which probably applies to most cases with some minor variations:

DESIGN FEATURES

1. Selection of the best general route:
   a. Best location for the development of the territory.
   b. Longest open season.
   c. Least rise and fall.
   d. Length and cost.
2. Selection of the most natural engineering location following the desired general route:
   a. Reasonable grades.
   b. Exposure. Avoid north exposure and areas of deep snow.
   c. Character of excavation. Avoid rock, slides, etc.
   d. Drainage problems. Avoid flood areas, stream crossings, etc.
   e. Avoid artificial restrictions, such as section-line locations, etc.
   f. Avoid needless railroad grade crossings.
3. Detail requirements of design:
   a. Reasonable maximum grade, considering future requirements.
   b. Economical intermediate grades.
   c. Safe and economical alignment, considering future requirements.
   d. Width of roadway safe for traffic, eliminating dangerous ditches.
   e. Width of roadway convenient for traffic.
   f. Sufficient culverts and bridges to protect the roadway, considering the future.
   g. Permanent construction of these culverts and bridges.
   h. Sufficient width of clearing for sun to reach road.
   i. Safety provisions. Protection for traffic at dangerous places.
   j. Provision of liberal width of right of way, considering future widenings and developments.
4. Improvement of the road surface:
   a. By selective soil treatment.
   b. By gravel, chert, macadam, etc.
   c. By rigid pavements.
5. Improvements for the future:
   a. A higher-grade surface.
   b. A wider hard surface.
   c. Provision of sidewalks for pedestrians.
   d. Planting trees, etc.
   e. Illumination.

An examination of the roads in almost any locality leaves the impression that a little more emphasis on and attention to the better construction of the fundamental features will add to the reasonable proportion of design and be a move in the right direction.

The following typical cases illustrate the usual problems that occur and indicate their general solution.

General Solutions, Pioneer Districts.—Where no road exists and the funds are entirely too small for good construction, a sufficiently cheap design is used to complete the entire length. Under these conditions the only requirement that must be met is the proper selection of general route, although it is probable that for the greater part of the distance the final engineering location can be followed. Considerable work of this kind has been done in the Southwestern states, and the solutions are ingenious. Satisfactory wagon and automobile trails have been constructed under favorable conditions for as low as $20 per mile, while in difficult locations advantage has been taken of all possible expedients to keep the cost down.

Where a poor but usable road exists between terminal points, or for a portion of the distance, either the uncompleted or the worst sections of the route are first considered. Under such circumstances the funds are generally sufficient to permit a moderately good engineering design, which must provide for a reasonably good grade and drainage scheme on the improved sections, although the drainage structures may be cheap and temporary and the roadway narrow.

Where a fair road has been previously built over the entire route, no improvement should be attempted unless it provides for a first-class engineering design of grades, alignment, section, and permanent drainage structures.

Where a first-class natural-soil road is in use, the next step in progressive improvement requires either selected soil, gravel, or hard-surfaced construction of the traveled way.

General Solutions, Well-settled Districts.—The application of the order of importance of design elements for hard-surfaced pavement work can be shown by three cases:

Under the most favorable conditions in rich communities, the improvement is considered final and its design is based on an effort to obtain the most useful, and in the end the most economical, form of construction regardless of the first cost. In this case all the engineering requirements may be fulfilled.

In many communities, however, the funds are only sufficient to build a moderately good pavement, which will have to be bettered by reconstruction in a few years, to meet the increasing demands of the traffic. An improvement of this kind should be permanently and completely designed for proper grades, alignment, section, drainage and safety provisions up to a certain reasonable limit and the balance of the money spent on the best type of hard surface that can be afforded.

The third case is reconstruction, which usually confines the problem to consideration of the most suitable type of resurfacing, utilizing previous work to the best advantage. It also sometimes involves improved relocation.

Reasonable Economy in Design.—The mileage to be constructed is so great and the amount of money involved so impressive that it seems desirable to use all reasonable care to produce as many miles of road as possible with the available funds. During the years 1913–1920 the author has made a careful review of some 2000 miles of road plans from different sections of the country with the idea of forming a reasonable conclusion as to the trend of highway design and to see how closely current practice follows the well-recognized principles of highway engineering. The results of the analysis of these plans were, roughly, as follows: About 25 per cent could be classed as first-class designs from any economical standpoint. Practically all the designs showed minor wastes, but for the plans classed as good, revisions would not result in any practical advantage. About 75 % cent of the plans showed a material expenditure of money for which no adequate return was obtained, amounting to from 5 to 20% of the cost. On some of the roads which, as built, served the traffic well, this excess might better have been spent on other jobs. On some of the roads, which, as built, were not up to the requirements of the traffic, the waste might better have been applied to their own improvement in fundamental features.

The general faults most noticeable were:

Too much spent on the reduction of intermediate grades.

Too much spent to obtain long, straight grades.

Too much spent on sections with deep ditches.

Not enough spent on realignment at dangerous locations.

Not enough spent on relocations necessary to get reasonable maximum grades.

Not enough spent on long-span bridges.

Too much spent on width of pavement.

Not enough spent on depth of pavement.

Too much spent on imported materials where local materials were available in limited quantities.

One of the objects of this book is to discuss in detail various proved means of effecting economies without reducing the usefulness of the roads. At this point, however, it is not necessary to do more than to indicate the different parts of design that are particularly susceptible to such saving.

Systematic grading design will often reduce the work from 500 to 2500 yd. per mile, amounting in money, on an average, to from $500 to $3000 per mile. The proper use of local material, particularly in foundations, is a large factor in economy and will often reduce the cost from $1000 to $3000 per mile. Reasonable variations in pavement width and in the thickness of surfacing courses is effective, and in many cases saves from $1000 to $2000 per mile. A very conservative estimate of savings due to these systematic minor alterations is from $1000 to $2000 per mile. These savings are not spectacular for any one job, but if consistently used their advantage on any large program is very evident. They will more than pay for all the necessary engineering work in connection with the entire program. The small additional work required for a careful analysis is the best possible engineering investment for the community that can be made.

Tests of Designs.—It is certainly well worth while to test out each finished design to see if it complies with the general principles which have been discussed and also with the detail economics that will be taken up later. The following list of questions indicates in a general way the points to be considered:

QUESTIONNAIRE

Is the alignment suitable for all reasonable requirements of the future?
Is the ruling grade suitable for all reasonable requirements of the future?
Is the section, ditch to ditch, safe and suitable for present traffic?
Is the right of way wide enough for future requirements?
Are there ample culverts for all requirements of the future?
Are the culverts proportioned properly as to size, considering run-off?
Are the culverts long enough to be safe and large enough to maintain?
Are the bridge superstructures strong enough for present traffic?
Have all permanent culverts and bridges been designed strong enough and wide enough for, say, 50 years?
Are the bridge abutments for new temporary superstructures solid enough for future permanent superstructures?
Are the ditches road ditches and not farm drainage ditches?
Are the safety provisions real safeguards or are they only warnings?
Is the road surface thick enough to handle present traffic without foundation failure, considering the subsoil conditions?
Is the road surface wide enough for present traffic?
Is the surface of the general type required by present traffic?
So much for proportion—now for economy:
Does the grade line conform with the principles of economical design?
Do the sections fluctuate to conform to economical design?
Has the selection of pavement type been based on the most economical use of local materials?
Has the design been varied to use limited supplies of local material with short hauls?
Is the width reasonable, and has it been varied on a road that has heavy traffic part of the distance and light traffic part of the way?
Has the depth of macadam been varied to meet the different requirements of the soils and kept to a reasonable minimum?
Have the culverts and bridges been designed for the most economical type for the span in question?
Have the types of culverts been varied to get the cheapest result, considering local materials, in comparison with market quotations and cost of long hauls on imported materials?
Are the specifications flexible enough to permit reasonable use of local material?
Does the testing laboratory make an effort to approve the reasonable use of local material, or is it inclined to hold arbitrarily to the highest standards, regardless of the relative importance of the job in hand?

The designer should, however, bear in mind that imperfections in construction and intermediate factors make too close a theoretical design impracticable and that a certain factor of safety must be provided in all his plans for such possibilities. The application of this to the different elements of design will be discussed throughout the book.

Economic Engineering Design.—Economic engineering design balances value against cost and considers the following factors:

1. Traffic classification (volume and kind).
2. Economical design and value of improved location, grades, and alignment.
3. Suitable pavement types, based on traffic classification.
4. Subgrade soils.
5. Economic design of alternative pavement types considering soil and traffic.
6. Local and imported material available.
7. Comparative cost estimates (alternative types):
    a. Construction.

   b. Maintenance.
   c. Renewal.
   d. Motor operation costs.
8. Reasonable maximum and minimum expenditures.
   a. Economic value of improvement based on reduction in travel cost.
   b. General intangible value.
9. Recommended design considering both technical and non-technical factors.

The economic designs of Bridges and Grade Crossings are discussed in Chapters 4 and 9.

1. *Traffic Classification.*—Estimates of present and future traffic are discussed on page 32 and general traffic classification on page 46.

2. *Economical Design and Value of Improved Location, Grades, and Alignment.*—The second part of the analysis considers a study of relocation, grade reduction, and alignment.

Practice in matters of alignment are based entirely on the factor of safety and no consideration is given to reduction in motor operation cost, for, while motor operation costs are increased by dangerous alignment, they are not greatly affected by minor improvements in relatively safe alignment. Dangerous alignment is eliminated on the basis of general or intangible benefit, which at the same time gives some economic cost benefit. (See Chap. II for the effect of alignment on cost.)

For deriving the approximate value of proposed grade reductions or relocations, Tables 5 and 6 (p. 12) give a rough approximation of the capitalized motor operation cost per foot of distance traveled by an average volume of 100 vehicles daily average mixed traffic for each rate of grade from level to 10%. Table 6 assumes one-half traffic up and one-half down each grade, and is very simple to use. In comparing proposed alternate locations or profiles using different rates of grade, it is necessary to compute only the sum total distance in feet of each rate of grade used on the different designs and to multiply by the money value given in the table, summing up the resultant products for each proposed design. The total difference multiplied by the volume of future traffic considered reasonable gives a quite rational basis for conclusions as to the relative value of the alternate designs considering motor operation costs. The relative cost of the alternate proposed routes are then balanced against their relative value. An example of such a tabulation is given on page 57.

The values for the different rates of grade range from $9 per foot per 100 vehicles daily to $127. The derivation of Table 6 has appeared in various publications. For the last 8 years it has been used by the author in connection with all grading designs for which he has been responsible. It shows quite plainly that the reduction of natural grades of 4% or less are rarely justified. For higher rates of grade each case is a special problem, three examples of which will be given to illustrate the value of an economic analysis of this kind.

The first example is the Woodville Bristol Springs Highway in the Finger Lake District of New York. This is a rough, hilly county and the original road had dangerous alignment and 15% grades. Two relocations were considered, one based on straight alignment and 10% grades which is the extreme maximum for such conditions, the other on 7% maximum grades, greater distance, and safe but crooked alignment. Each of these locations had certain backers and the road was deadlocked. An economic analysis was applied based on the preceding data. This analysis showed that the value of the 7% location from the standpoint of motor operation probably more than justified the increased cost of this location. The 7% was adopted.

The second example refers to the design of approach grades to a highway bridge over a railroad crossing. This crossing occurs on a relatively unimportant state road. The following is quoted from the recent official report where Table 6, page 12 was used in figuring the economics of the design.

COMPARISON CAPITALIZED MOTOR OPERATION COST, OLD AND NEW PROFILE (ROAD 1392)
(Based on column 1, Table 6, page 12)

Rate of grade, %	Estimated capitalized cost per 100 vehicles daily per foot of distance	Old profile		New profile	
		Length, feet	Amount 100 vehicles daily	Length, feet	Amount 100 vehicles daily
1.0 or less	$ 9.10	8,300	$75,530	7,900	$71,890
1.5	9.12	800	7,296	400	3,648
2.0	9.15	1,200	10,980	1,700	15,555
2.2	9.17	200	1,834	800	7,336
2.4	9.19	500	4,595	700	6,433
2.6	9.21	600	5,526	300	2,763
2.8	9.23	1,000	9,230	800	7,384
3.0	9.25	800	7,400	1,600	14,800
3.2	9.28	800	7,424	500	4,640
3.4	9.31	500	4,655	500	4,655
3.6	9.34	400	3,736	1,000	9,340
3.8	9.37	100	937		
4.0	9.40	400	3,760	700	6,580
4.2	9.45	300	2,835	200	1,890
4.4	9.50	400	3,800	400	3,800
4.6	9.56	500	4,780	100	956
4.8	9.63	200	1,926	200	1,926
5.0	9.70	300	2,910	100	970
5.2	9.80	100	980	100	980
5.4	9.90	100	990		
5.6	10.00	400	4,000	300	3,000
5.8	10.10	400	4,040	400	4,040
6.0	10.20	.....	......	800	8,160
6.5	10.70	.....	......		
7.0	11.20	100	1,120		
7.5	11.85	500	5,925		
8.0	12.50	200	2,500		
8.5	13.25	100	1,325		
9.0	14.00	100	1,400		
9.5	14.75	.....	......		
10.0	15.50	200	3,100		
100 vehicles daily total			$184,534	.....	$180,746
500 vehicles daily total			$922,000	.....	$904,000

Maximum economic value of grading $18,000. ($922,000–$904,000)

"Construction cost is increased but motor operation cost decreased by the adoption of lower rates of grade. The use of a 7% approach grade advocated by Mr........... instead of the 8.5% recommended would increase the construction cost $14,000 over the cost of the 8.5%. While it is impossible to figure the capitalized cost of motor operation exactly, it is not likely that the reduction in operation cost would warrant an increase in construction cost of over $6,000, assuming 300,000 cars yearly, which is a liberal allowance for future traffic. The increased cost of construction is out of all proportion to the benefit derived, which indicates that 8.5% grade is preferable to 7.0% from the standpoint of economic benefit."

The 8 5% rate was adopted.

The third example covers the use of these data in connection with deriving the value of a proposed highway improvement. The following is quoted from the official report on Road 1392, which is a Class III crossroad on the state system.

"The existing grades are, in the main, easy, with a few short steep grades of 8 to 9%. The new profile, as a rule, follows the existing road closely, with short grade reduction using 6% maximum grades. The total rise and fall for the old and new road is about the same. The proposed grading will have some effect in reducing motor operation costs, but is largely useful in providing a convenient width for traffic and a smooth bed for the pavement. In figuring the economic value of the improvement grade reductions can be given some weight for this road. Total grading cost estimated at $32,000. Total economic benefit of grading estimated at $18,000, allowing 200,000 vehicles yearly, which is liberal for this highway" (see tabulation page 57 for detail method).

3. *Suitable Pavement Types Based on Traffic Classification.*—The discussion of this part of the design was given on page 46. Table 13 gives the summarized results.

TABLE 13.—GENERAL SUITABILITY OF TYPES

Class I roads, special-service roads, 2000 or more vehicles daily (10-hr. count in summer)	Class II roads, ordinary mixed traffic, 800 to 2000 vehicles daily (10-hr. count in summer)	Class III roads, ordinary mixed traffic, 300 to 800 vehicles daily (10-hr. count in summer)	Class IV roads, local traffic less than 300 vehicles daily (10-hr. count in summer)
Asphalt block on concrete base Brick block on concrete base Bituminous concrete on concrete base Reinforced cement concrete Stone[1] block on concrete base	Bituminous[2] concretes on old firm macadam base Small block pavements with flexible joint filler on old firm macadam base Reinforced cement concrete Penetration bituminous macadam	Bituminous macadam Water-bound macadam (oiled) Bituminous gravel Gravel (oiled)	Macadam (oiled) Gravel (oiled) Sand clay in districts not subject to severe winters
WIDTHS OF PAVEMENT[3] (In feet)			
18–20, with additional width of macadam shoulders	15–18, with additional stone shoulders	12–16	8–12

[1] Special conditions only.
[2] The use of these types on newly built macadam is not safe practice.
[3] Extra width on sharp curves.

4. *Subgrade Soils.*—The character of the subgrade soil controls the pavement design: While the authors have a great respect for recent soil research, they do not consider it practicable to make laboratory analyses of soil samples. The procedure advocated and carried out on all roads for which they have been responsible is a rough classification, station by station, made by their most experienced field men., They interview each property owner and road commissioner for the location of soft spots and investigate the character of the soil for the entire length of the road by examination of existing cuts and by bar soundings and gas pipe core samples, where necessary, for depths to at least 3′ below the proposed grade of the improvement. The results are tabulated under the following general soil classification, which are considered sufficiently accurate for all practical purposes in designing new pavements: Gravel, Coarse Sand, Sandy Loam, Loam, Clay Loam, Ordinary Clay, Heavy Wet Clay, and Quicksand.

For reconstruction designs of old macadam roads the depth of the existing macadam is determined and also its general condition, whether firm or mixed with the underlying soil. The underlying soil is classified as for new construction.

5. *Economic Design of Alternate Pavement Types, Considering Soil and Traffic.*—In order to make rational comparative estimates of final highway costs, it is necessary to adopt adequate pavement depths for the different types. Pavement strength affects not only construction cost, but also maintenance and renewal costs. Inadequate strength reduces first cost, but increases maintenance and renewal. The authors' experience for the last 20 years indicates that the pavement depths given in the attached tables are adequate, and result in economical final cost, including maintenance and renewal. These depths are affected by the subsoil and traffic. The depths recommended consider not only the soils but whether the pavements are in cut or on high fills.

They are based on maximum vehicle loads of 28,000 lb., for year-round travel, Class I and II highways, reduced to 12,000 lb. during the spring thaw for macadam roads on Class III and IV highways.

The macadam depths range from 6 to 30″ and the rigid-pavement depths from 6 to 13″, with additional gravel or stone subfoundations where necessary (see Tables 14 and 15).

Pavement Widths.—Pavement width depends, first, on safety and convenience, and, second, on cost. Experience indicates that for traffic of less than 300 daily, single-track pavements 8 to 12′ wide with a grading width of 20′ serve satisfactorily. For Class III traffic (300 to 800 daily), normal widths of 12 to 16′ are moderately safe and convenient, with a grading width of 22′ for turn-out traffic. Extra width on sharp curves is provided for all roads of Classes III, II, and, I (see p. 132). For Class II traffic (800 to 2000 daily), normal widths of 16 to 18′ with special gravel shoulders are both safe and economical from a maintenance standpoint. On Class I traffic (over 2000 daily), 18 to 20′ normal widths with special macadam shoulders serve well up to about 6000 vehicles in 12 hr. For volume of traffic above this count, the construction of parallel routes is advocated, to reduce congestion, but where this is not

TABLE 14.—RECOMMENDED TOTAL DEPTHS OF FLEXIBLE PAVEMENTS IN LOCALITIES SUBJECT TO SEVERE WINTERS (IN INCHES)

Class I Traffic (over 2,000 vehicles daily, 10-hr. count in summer)
(Macadam not usually economical on Class I roads)

Soil	Location of road		
	In cuts or on shallow fills less than 1 ft. deep	On intermediate fills 1 to 3 ft. deep	On high fills over 3 ft. deep
Coarse sand and fine gravel	9 to 10	9 to 10	9
Loams	10 to 14	10 to 12	10
Ordinary clays	15 to 21	12 to 16	11
Heavy clays and fine sands	22 to 30	14 to 18	13
Class II Traffic (800 to 2,000 vehicles daily, 10-hr. count in summer)			
Sand and gravel	8 to 9	8 to 9	8
Loams	9 to 12	9 to 11	9
Ordinary clays	12 to 18	11 to 15	10
Heavy clays and fine sands	18 to 28	13 to 16	12
Class III Traffic (300 to 800 vehicles daily, 10-hr. count in summer)			
Sand and gravel	7 to 8	7 to 8	7
Loams	8 to 10	8 to 9	8
Ordinary clays	12 to 16	9 to 14	9
Heavy clays and fine sands	18 to 24	12 to 15	11
Class IV Traffic (less than 300 vehicles daily)			
Sand and gravel	6 to 7	6 to 7	6
Loams	7 to 9	7 to 8	7
Ordinary clays	10 to 15	8 to 12	8
Heavy clays and fine sands	15 to 22	10 to 14	9

possible 27 or 36′ widths of pavement are used. As a general rule, it does not seem wise to cater to the policy of concentrating traffic on a few roads and it is believed better to limit pavements to the 20′ width for single roads except near cities where unavoidable concentration occurs. For discussion of road capacity see page 28. For roads carrying more than 1500 vehicles daily the shoulders should be wide enough to permit all cars which are parked or stopped for repair to be entirely off the pavement proper.

The consideration of width completes the factors of design which control the quantities for comparative estimates of pavement cost.

6. *Materials Available.*—The local and imported materials control unit cost estimates. Investigations of the sources of supply are routine procedure and no special comment is required in regard to this factor, except that it is not wise to demand the same standard

of excellence for materials on roads carrying less than 800 daily as it is for highways carrying over that amount.

7. *Comparative Pavement Cost Estimates.*—Comparative cost estimates include the cost of construction, maintenance, and renewal. Construction cost estimates for different types of pavement suitable to the general class of traffic the improvement will serve is the next step in procedure. These estimates are based on the design data previously discussed and on the selection of the cheapest source of material supply suitable for each type of pavement. For western New York macadam pavements of adequate depth cost from \$1.50 to \$3.80 per square yard and rigid pavements of adequate strength from \$2.30 to \$5 per square yard, depending on soils, materials, and traffic. It is very evident that the wide range makes a careful analysis necessary to determine which type has the advantage for any specific case.

Maintenance costs shown in Table 16 are based on general records for the last 20 years modified by consideration of selected roads which approximate the modern design standards previously discussed and which have received proper maintenance. The attached table of maintenance cost, represents what is considered a fair average for properly designed pavements during the period recommended as reasonable economic life before renewal becomes necessary. These costs do not agree exactly with reported general averages, as a large percentage of existing mileage is entirely too weak for modern traffic and also a considerable mileage of rigid pavements has been in service for such a short time that maintenance charges are less than normal.

In a general way, the maintenance of well-designed and maintained macadams will range from 1.5 to 7 cts. per square yard, per year, depending on traffic. The cost of rigid pavement maintenance over a long term of years will probably run between 0.5 to 3 cts. per square yard per year, depending on the type and the traffic. These costs do not include incidental maintenance of shoulders, ditches, etc., which are considered as a separate item.

The average yearly renewal costs are based on western New York records for the past 20 years modified by the fact that funds for reconstruction have lagged behind the necessities of the situation, which results in low reported costs and extremely rough highways during the last few years before reconstruction. The renewal costs given are based on probable length of life, during which the pavements will not become disagreeably rough (250″ per mile Vialog rating) without excessive maintenance (\$1000 per mile year maximum). Under these conditions the life of macadam pavements is estimated between 5 to 17 years with an average yearly renewal cost of between 6 to 20 cts. per square yard, depending on the traffic. The life of rigid pavements will range from 8 to 30 years, with renewal costs of between 7 to 17 cts. per square yard.

Motor Operation Costs.—Consideration of the relative cost of motor operation over the different types of pavement completes the economic comparison of alternate designs.

There is no possibility of deriving exact costs of operation over the different types, as there is a wide range in power used, tire wear,

TABLE 15.—COMPARISON OF RECOMMENDED THEORETICAL DEPTHS OF CEMENT CONCRETE BASES FOR DIFFERENT PAVEMENTS UNDER DIFFERENT MAXIMUM LOADS ON DIFFERENT SOILS (IN INCHES)

NOTE.—The last column gives prevailing practice in base depth for each type (1919 to 1922).

Pavement	Recommended depths of cement concrete bases based on modified corner load formula $d = P\sqrt{\frac{JW}{S}}$						Current practice 1919–1923, inches
	3½-ton truck 16,000-lb. gross load		5-ton truck 22,000-lb. gross load		7-ton truck 28,000-lb. gross load		
	Ordinary subsoils, inches	Gravel or macadam, inches	Ordinary subsoils, inches	Gravel or macadam, inches	Ordinary subsoils, inches	Gravel or macadam, subsoils, inches	
Plain concrete:							
(1:1½:3 mix)	7.2 & 7.8	6.5 & 7.0	7.9 & 8.8	7.2 & 8.0	8.7 & 9.7	8.0 & 8.8	6 to 10
Plain concrete:							
(1:2:4 mix)	7.6 & 8.2	6.9 & 7.4	8.4 & 9.2	7.6 & 8.3	9.2 & 10.0	8.4 & 9.1	
Reinforced concrete (mesh and bar):							
Central longitudinal joint (1:1½:3 mix)	6.3 & 6.9	6.0 & 6.4	6.7 & 7.5	6.2 & 6.8	7.3 & 8.2	6.5 & 7.3	
Reinforced concrete (corner and exterior tie bars only):							
Central longitudinal joint 1:1½:3 mix	6.5 & 7.0	6.0 & 6.4	7.0 & 7.8	6.3 & 7.1	7.7 & 8.5	7.0 & 7.7	5 to 9
1:2:4 mix	6.9 & 7.4	6.4 & 6.8	7.4 & 8.2	6.7 & 7.4	8.1 & 9.0	7.4 & 8.1	
Monolithic brick 4-in. brick on 1:1½:3 concrete base	4.5	4.0	5.0	4.5	6.0	5.5	4.0
2-in. to 3-in. *Asphaltic concrete on concrete base:*	6.5	6.0	7.0	6.3	7.5	6.8	5 to 8
(1:3:6 mix)	6.0	6.0	6.5	6.0	7.0	6.3	
(1:2½:5 mix)							
2-in. *Asphalt block on cement concrete base:*							
(1:2½:5 mix)	6.5	6.0	7.0	6.3	7.5	6.8	5 to 8
4-in. *Brick* (mastic joint filler) *on cement concrete base:*							
(1:3:6 mix)	6.8	6.2	7.6	6.9	8.3	7.5	
(1:2½:5 mix)	6.5	6.0	7.0	6.3	7.8	7.1	6 to 8
4-in. *Brick (cement grout filler):*							
(1:3:6) concrete base	5.5	5.0	6.1	5.5	6.7	6.2	5 to 8
(1:2½:5) concrete base	5.2	5.0	5.7	5.2	6.3	5.7	
5-in. *Stone block: Cement grout filler:*							
(1:2½:5) concrete base	5.2	5.0	5.7	5.2	6.3	5.7	5 to 7

NOTE—See bottom page 63 for explanatory note.

etc. of a vehicle traveling over any type of pavement due to the relative perfection of the surface, which is affected by excellence of construction, age of pavement, and excellence of maintenance regardless of the type of pavement being investigated (see Figs. 8 and 9).

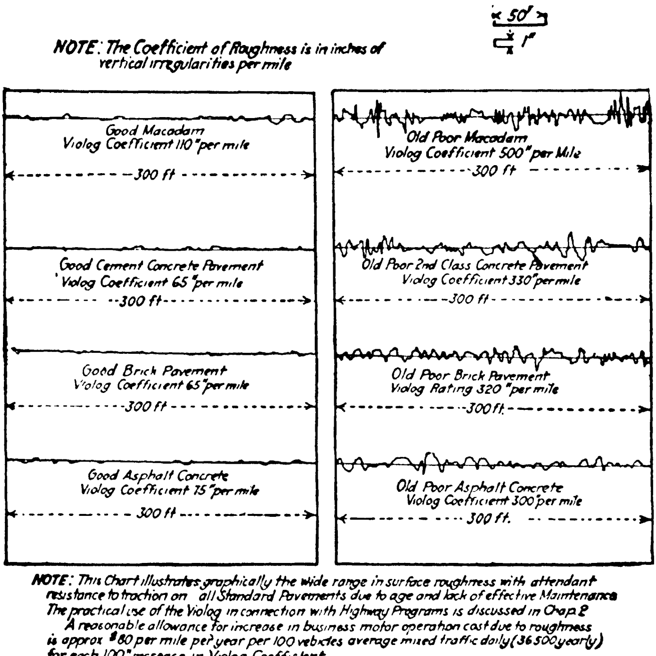


FIG. 8.—Vialog records of surface roughness (showing effect of age).

It is, however, possible with the data at our disposal (1926) to compare the cost of vehicle operation over the two general classes of pavement surfaces in common use and arrive at general conclusions, although the range in possible values is quite large. For

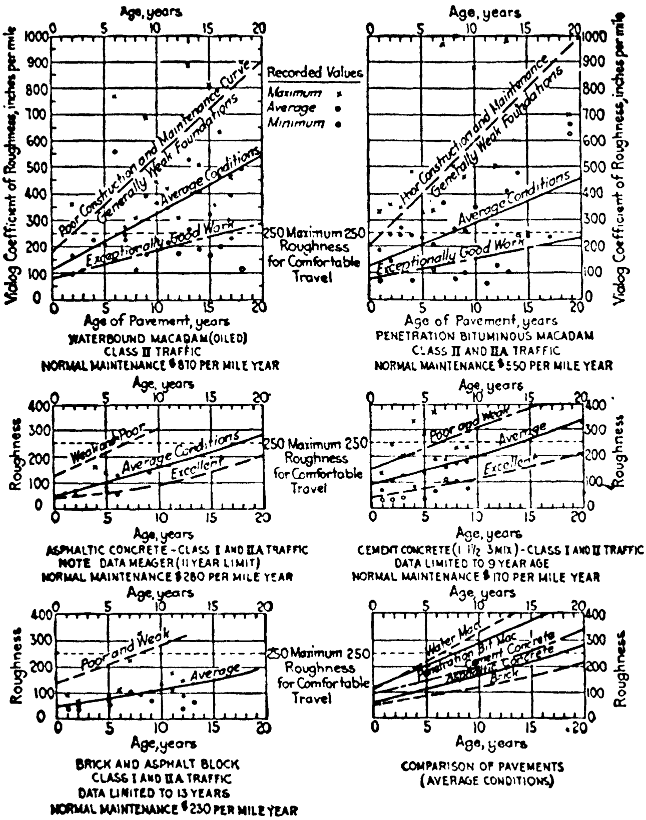
(text continued on page 67.)

NOTE TO TABLE 15

NOTE: Where two depths are given the smaller is for interior areas and the larger for exterior edge depths. Where one is given it applies to interior areas.

Typical recommended Pavement Sections showing details are given in Chap. VI under the discussion of each type of pavement. The object of this table is to give a tentative basis of estimating the amount of materials and the cost per square yard for the different types suitable for different maximum load conditions.

Table 15 in conjunction with Table 14, page 60, provides a means of computing reliable comparative cost estimates for all ordinary standard types based on equal strength.



GRAPHS SHOWING WIDE RANGE AND AVERAGE VALUE OF VIALOG CO-EFFICIENT OF ROUGHNESS FOR DIFFERENT PAVEMENTS AT DIFFERENT AGES.
(NEW YORK STATE 1923)

Based on average maintenance, that is, no specially high maintenance has been used to hold the roughness to a specified maximum Reconstruction is resorted to when normal maintenance becomes ineffective.

NOTE: *These graphs show the wide range in results obtained under Public Works Programs and show conclusively the distinct advantage of good Engineering Control Any type of pavement well designed, constructed and maintained will give satisfactory results as far as traffic operation cost is concerned.*

To anyone personally familiar with this district these records have a human as well as a scientific interest. It is possible to trace the gradual loss or gain in interest in the success of the various types depending on the sympathies of the different administrations and they also show the effect of knowledge and experience of the inspectors with different types.

FIG. 9.

TABLE 16.—PROBABLE AVERAGE MAINTENANCE AND RENEWAL COSTS FOR DISTRICTS SIMILAR TO WESTERN NEW YORK (1922)

Computed for normal conditions. Exceptionally favorable conditions in regard to excellence of original construction and to maintenance appropriations and personnel might easily reduce maintenance and renewal costs 20 to 30 per cent (see Table 105, Chap. VII).

Type of pavement	Width, feet	Average depth, inches	Construction costs		Yearly maintenance	
			Contract cost per square yard pavement only	5 Per cent interest on cost	Pavement only per square yard	Shoulders, ditches, guard rail, etc.
Column No. 1	2	3	4	5	6	7
Class 1 Traffic (2000 or more Vehicles Daily 10-hour Count in Summer)						
Stone block on concrete base	18–20	12	$5.50	$0.275	$0.005	$0.020
Brick on concrete base	18–20	11	4.20	0.210	0.015	0.020
Asphaltic concrete on cement base	18–20	10	3.50	0.175	0.020	0.020
Reinforced cement concrete	18–20	8	3.20	0.160	0.015	0.020
Penetration bituminous macadam[1]	18–20	13	2.50	0.125	0.070	0.020
Class 2 Traffic (800–2000 Vehicles Daily 10-hour Count in Summer)						
Reinforced cement concrete[1]	16–18	8	$3.20	$0.160	$0.010	$0.015
Asphaltic concrete on macadam base	16–18	12	3.00	0.150	0.020	0.015
Brick cubes on macadam base	16–18	12	3.00	0.150	0.020	0.015
Penetration bituminous macadam	16–18	12	2.20	0.110	0.040	0.015
Waterbound macadam (oiled)	16–18	12	1.90	0.095	0.080	0.015
Class 3 Traffic (300 to 800 Vehicles Daily 10-hour Count in Summer)						
Penetration bituminous macadam	12–16	10	$1.90	$0.095	$0.030	$0.012
Waterbound macadam	12–16	10	1.60	0.080	0.060	0.012
Class 4 Traffic (Less than 300 Vehicles Daily)						
Waterbound macadam	8–12	9	$1.50	$0.075	$0.040	$0.012
Gravel[2]	8–12	10	1.00	0.050	0.040	0.012

[1] These types eventually resurfaced with asphaltic concrete or some form of standard block or cube surface.

[2] This type can be temporarily used under heavy traffic but for a volume of over 400 to 500 daily it generally develops disagreeable waves known as rhythmic corrugations.

TABLE 16.—PROBABLE AVERAGE MAINTENANCE AND RENEWAL COSTS FOR DISTRICTS SIMILAR TO WESTERN NEW YORK (1922)—*Continued*

Computed for normal conditions. Exceptionally favorable conditions in regard to excellence of original construction and to maintenance appropriations and personnel might easily reduce maintenance and renewal costs 20 to 30 per cent (see Table 105, Chap. VII).

Type of pavement	Renewal[3]		Total maintenance and renewal		Total interest on construction, maintenance and renewal	
	Assumed normal life	Yearly cost for renewal	Per square yard	Per mile	Per square yard	Per mile
Column No. 1	8	9	10	11	12	13
Class 1 Traffic (2000 or more Vehicles Daily 10-hour Count in Summer)						
Stone block on concrete base	30–35	$0.100	$0.125	$1400	$0.400	$4400
Brick on concrete base	15–20	0.160	0.195	2150	0.405	4450
Asphaltic concrete on cement base	10–15	0.170	0.210	2300	0.390	4300
Reinforced cement concrete	10–15	0.160	0.195	2150	0.360	4000
Penetration bituminous macadam[1]	5–9	0.206	0.290	3200	0.420	4600
Class 2 Traffic (800–2000 Vehicles Daily 10-hour Count in Summer)						
Reinforced cement concrete[1]	15	$0.145	$0.170	$1,700	$0.330	$3,300
Asphaltic concrete on macadam base	12	0.150	0.185	1,850	0.335	3,350
Brick cubes on macadam base	12	0.160	0.195	1,950	0.345	3,450
Penetration bituminous macadam	10	0.120	0.175	1,750	0.285	2,850
Waterbound macadam (oiled)	8	0.110	0.205	2,050	0.300	3,000
Class 3 Traffic (300 to 800 Vehicles Daily 10-hour Count in Summer)						
Penetration bituminous macadam	12	$0.100	$0.140	$1,100	$0.240	$1,900
Waterbound macadam	10	0.070	0.140	1,100	0.220	1,800
Class 4 Traffic (Less than 300 Vehicles Daily)						
Waterbound macadam	12	$0.060	$0.110	$ 650	$0.180	$1,100
Gravel[2]	8	0.030	0.080	500	0.130	800

[1] These types eventually resurfaced with asphaltic concrete or some form of standard block or cube surface.

[2] This type can be temporarily used under heavy traffic but for a volume of over 400 to 500 daily it generally develops disagreeable waves known as rhythmic corrugations.

[3] Based on maximum vialog coefficient of 250 in. per mile without excessive maintenance charge.

purposes of comparison, the ordinary rural pavements are classed as follows:

Class A. Brick, bituminous concretes, and Portland cement concrete.

Class B. Bituminous macadam and water-bound macadam or gravel with bituminous surface coats.

Table 4 (p. 11), based on investigations at the Iowa State College in conjunction with U. S. Bureau of Roads, shows a difference in average vehicle operating cost of approximately 0.5 ct. per mile, or 5% of the total operating cost of vehicles in favor of Class A surfaces as compared with Class B. This is considerably in excess of the authors' data.

The authors' investigations as published in "Rural Highway Pavements" in connection with Vialog records in western New York indicate that, making due allowance for average roughness over the total life of the pavement, snow and ice in winter, etc., the difference in operating costs for well-maintained Class A and B surfaces does not probably exceed 3% of the cost of gasoline, oil, tires, and repairs or approximately 0.14 cts. per average vehicle mile. (See Chap. VII, p. 545). As discussed on page 10 business travel is approximately 50% of total travel which must be considered in computing the actual loss to the community at large from the use of Class B surfaces.

The following tabulation indicates roughly the loss per mile of highway per year for different volumes of travel and the capitalized value of this yearly loss at 5%, which represents the added justifiable cost of Class A surfaces over the cost of Class B surfaces due to difference in vehicle operating costs. This tabulation is based on the authors' figures of 0.14 cts. given above. The following quotation strengthens the discussion.

"In computing the total yearly cost of a pavement to the community it is just as well to give some weight to the item of reduction in operating cost.

"For average conditions in Division N. 4, western New York, considering construction, maintenance, renewal, interest on construction, and difference in motor operation cost, the general conclusion to be drawn is that the macadam type of surface usually becomes uneconomical from a business standpoint at a volume of traffic of between 1000 to 2000 vehicles daily. At this point an old macadam road can be resurfaced with asphaltic concrete and brought up to the operation efficiency of Class A pavements."

Examples of Economic Comparisons of Pavements.—To illustrate definitely the use of the foregoing data in comparing the economic value of different pavements, considering construction maintenance, renewal, and motor operation costs, two examples from recent design reports will be cited.

Road 1392.—This road is a Class III traffic crossroad on the state system. For purposes of economic comparison a volume of traffic of 500 daily average has been adopted and the use of three alternate pavement designs considered· water-bound macadam, oiled, bituminous macadam, and reinforced cement concrete. A 16′ width has been adopted, as, while this may not be justified by the normal volume of local traffic, this road will be subjected at inter-

TABLE OF INCREASED VEHICLE OPERATION COSTS, CLASS B PAVEMENTS AS COMPARED WITH CLASS A PAVEMENTS

Number vehicles per day		Increased cost of yearly vehicle operation				Capitalized value of increased cost			
		Per mile		Per square yard 18′ pavement		Per mile		Per square yard 18′ pavement	
Total	Business only	Total	Business only	Total	Business only	Total	Business only	Total	Business only
100	50	$ 50	$ 25	$0.005	$0.002	$ 1,000	$ 500	$0.10	$0.05
500	250	250	125	0.025	0.012	5,000	2,500	0.50	0.25
1,000	500	500	250	0.05	0.025	10,000	5,000	1.00	0.50
2,000	1,000	1,000	500	0.10	0.05	20,000	10,000	2.00	1.00
3,000	1,500	1,500	750	0.15	0.075	30,000	15,000	3.00	1.50
4,000	2,000	2,000	1,000	0.20	0.10	40,000	20,000	3.90	1.95
5,000	2,500	2,500	1,250	0.24	0.12	50,000	25,000	4.80	2.40

vals to detour traffic from Route 6, the main east-and-west route. The economic comparison of types is tabulated as follows:

ECONOMIC COMPARISON OF TYPES, ROAD 1392

Type	Estimated construction cost per square yard	Interest on initial cost at 5%	Estimated yearly maintenance per square yard	Estimated average yearly renewal per square yard	Increased motor operation, Class B, per square yard	Total yearly cost, per square yard
Cement concrete	$2 30	$0.115	$0.010	$0.11	$0.00	$0.235
Bituminous macadam	1.60	0.080	0.035	0.11	0.01	0.235
Water-bound macadam	1 50	0.075	0.065	0.10	0.01	0.25

NOTE.—For conditions prevailing on this road there is very little difference in final economy for these three types.

The second example used is the Mt. Morris Village Highway, which is Class I, Stas. 26 to 48, and Class II, Stas. 0 to 26. The economic analysis given below shows an advantage for bituminous macadam from Stas. 0 to 26 and for cement concrete from Stas. 26 to 48.

COMPARISON, STAS. 0 TO 26

Type	Estimate cost of pavement, square yards	Yearly interest on construction square yards	Yearly maintenance, square yards	Renewal, square yards	Motor operation	Total yearly cost, square yards
Bituminous macadam	$1.60	$0.08	$0.035	$0.12	$0.015	$0.25
Cement concrete	2.70	0.135	0.01	0.14	0.00	0.285

COMPARISON, STAS. 26 TO 48

Type	Estimated cost of pavement, square yards	Yearly interest on construction, square yards	Yearly maintenance, square yards (Table 16)	Renewal, square yards (Table 16)	Motor operation	Total yearly cost, square yards
Bituminous macadam	$1.90	$0.095	$0.05	$0.16	$0.03	$0.335
Cement concrete	2.70	0.135	0.01	0.14	0.00	0.285
Asphalt concrete	3.10	0.155	0.02	0.15	0.00	0.325
Brick	4.10	0.205	0.015	0.13	0.00	0.350

While these analyses of final cost have some bearing on type selection, it is very evident from the small differences in total final cost, which are in many cases invalidated by poor construction or maintenance procedure, that type selection is more affected by the necessities of the situation expressed in terms of limitation of funds, initial cost of construction, and local preferences for certain types than it is by total final yearly cost; that is reasonable maximum and minimum expenditures for construction in a given case should be based on common sense rather than solely on an analysis of this nature.

8. *Reasonable Maximum and Minimum Expenditures and Recommended Design, Considering Both Technical and Non-technical Factors.*—In deciding on maximum and minimum expenditures, both direct reduction in motor operation cost and indirect intangible community benefits are considered. In computing the direct saving due to the improvement 1 ct. per ton-mile is a common allowance (see p. 14) of the value of our proposed pavement improvements. No allowance is made for minor differences between modern macadams or rigid-base pavements. To convert this direct yearly saving into permissible construction cost, multiply the total yearly saving in motor operation cost by 10. This procedure is based on maintenance and renewal records (Table 16) which show that the total yearly cost including interest on first cost, maintenance and renewal depends more on volume of traffic than on type of pavement within the general classes of traffic adopted and that it amounts to from 8 to 11% of the initial cost of construction.

On this basis Table 7 (p. 15) was derived. This table gives a reasonably close basis for estimating permissible average maximum expenditures based on travel cost saving due to pavement construction. To get the total permissible cost, add the value of grading and relocation to the pavement benefit.

Intangible general community benefits are more indefinite. Intangible benefits are expressed in terms of a rise in the general living standards of the community and, while they cannot be given an exact money value, a very definite idea of the types of road which must be constructed in order to obtain these benefits is derived. General community benefits can be obtained by what are called general-utility highways, which permit year-round travel for the normal business of the community with moderate comfort and with excessive maintenance cost (see p. 6).

Extreme refinements of grade or pavement surface which bring motor operation cost to a minimum are no factor for such roads. General-utility roads include all types of pavement, depending on the volume of traffic, and cost from $3000 to $60,000 per mile exclusive of bridges and grade-crossing eliminations. The usual costs for different volumes of traffic are given in Fig. 3 (p. 16). These costs are minimum permissible expenditures for any particular road and control expenditure where the direct reduction in travel cost does not permit an increase over these amounts.

To illustrate definitely the application of these principles, two examples are given, the first from official report on Road 1392, and the second from Mt. Morris village.

Example 1. *Road* 1392. "*Reasonable Maximum Expenditure.*—Reasonable expenditures based on reduction in traffic operation cost can be estimated approximately as follows:

"Grade reduction value (Art. 3 of this report, see p. 58) is estimated at not to exceed $18,000.

"Pavement value is roughly estimated as follows: The old existing road is a narrow gravel road in fairly good condition; the new pavement will not probably reduce the cost of operation over 1 ct. per vehicle mile, but it will add materially to the safety and convenience of operation, which are classed as intangible benefits. Actual reductions in operation cost (see Table 7) will not probably justify construction expenditures of over $16,000 per mile for 500 vehicles daily, or a total of approximately $60,000 for the entire length of this road. The total maximum economic value, including grading and pavement, does not probably exceed $75,000. Any expenditure over this amount must be charged to convenience, pleasure, and other desirable intangible benefits.

"This indicates that caution should be exercised in running the cost up needlessly.

"Total estimated construction costs for different widths of different types are tabulated below:

Type of pavement	Widths of pavement					
	8′	10′	12′	14′	16′	18′
Water macadam	$70,000	$78,000	$ 85,000	$ 99,000	$ 96,000	$105,000
Bituminous macadam	73,000	80,000	88,000	94,000	102,000	109,000
Cement concrete	85,000	96,000	108,000	110,000	128,000	138,000

"In order not to exceed the economic limit of $75,000 based on reduction of travel cost, we would be limited to a 10′ width of water-bound macadam oiled. A pavement as narrow as this is not in accord with our general policy, which limits the width to 16′, on the score of intangible benefit where the proposed road will at times carry detour traffic of a main route.

"We, therefore, recommend the use of 16′ width of bituminous macadam, estimated to cost $100,000. This recommendation seems to be warranted, as this type will satisfy traffic demands without running the construction cost up needlessly and without increasing the total final cost of road, including maintenance, renewal, and motor operation over and above the final cost from the use of a more expensive first-cost pavement.

"REMARKS.—Actual procedure on this road was as follows: Plans were first prepared for cement concrete pavement. Before these were let, the administration changed and the plans were revised for bituminous macadam, 7″ boulder base, 3″ imported slag middle course, and 2½″ imported limestone top; engineer's estimate of cost, $123,000. This road was constructed on this basis for $102,000 actual contract cost; the pavement proper cost $65,000, or approximately $1.75 per square yard. This final solution was an improvement over the first plans. A careful engineering design could probably have decreased the cost at least $8000 to $10,000 as shown in the body of this report by a more complete utilization of local materials and more variation in depth to meet varying soil conditions and a somewhat more economical grading design."

Example 2. *Mt. Morris Village* (see p. 69).—"From the standpoint of both original and final cost there is a every evident advantage for macadam from Stas. 0 to 26. The local people have no particular preference as to the type of pavement for this portion of the road. Local preference would be the only factor entitled to modify decisions based on the cost analysis, provided the additional cost is paid locally. As this factor need not be considered, we recommend bituminous macadam pavement from Stas. 0 to 26.

"From Stas. 26 to 48, local preference must be considered. The village proposes to pay the additional cost introduced by such preference as to type and extra width desired. Under these conditions the cost analysis is of value only as indicating proper state cooperation. Article 10 gives alternate total estimates of cost to enable the village to come to a reasonable decision as to the type they will ask for."

Recommended State and County Cooperation Based on Final Cost (Art. 7).— "The state would be justified in limiting their cooperation to the cost of a 16′ bituminous macadam pavement from Stas. 0 to 26 and an 18′ reinforced concrete pavement from Stas. 26 to 48 and 880 to 883 plus with ordinary surface-drainage provisions. This would amount to approximately $35,000. This is not an excessive amount if the funds are available (see Table 7), considering the volume of traffic served (about 1500 daily).

"I understand the rules of the Department limit cooperation to the construction of a 16′ road of the same type already constructed outside of the village. This rule would limit cooperation to a 16′ bituminous macadam which would cost approximately $30,000 for this locality. Under these circumstances it seems desirable to set the cooperation at $30,000.

"REMARKS.—Actual procedure on this road was as follows: The contract was let for reinforced cement concrete pavement from Stas. 0 to 26 and the brick pavement from Stas. 26 to 48, the local people paying the excess cost over and above the state fund limit advised in the foregoing report. This was a rational solution, considering local preference and willingness to assume added cost."

These examples complete the description of procedure and the practical part that economics play in design. The discussion shows quite plainly that economic analyses are desirable but that they are only one element in the decision.

Conclusion of Chapter.—This chapter may be briefly summarized as follows:

1. From the standpoint of economic return on road investments, it is desirable to limit the type of improvements to standards of general utility until a general system has been accomplished, particularly if the program is financed by a general tax levy.

2. Efficient maintenance of both the old existing roads and the new improved roads is the only possible means of giving moderately good general traffic service within a reasonable time.

3. It is probably desirable in most cases to begin by concentrating effort and at least 50% of the total available highway expenditure on a rapid construction program for certain main roads constituting say, from, 5 to 15% of the total road mileage. Such roads should, if possible, be built up to truly economic engineering standards of general utility.

4. After this is accomplished, it generally seems desirable to raise the standard of the entire balance of the system as can best be done. This may require low, makeshift engineering standards of pavement design for the first stages of improvement, depending entirely on the financial strength of the district. Under a reasonable tax distribution it is generally possible to improve the system gradually by successive stages up to the standards of general utility, provided traffic is subjected to reasonable regulation.

5. Unrestricted traffic makes it impossible to make any progress with local service roads in the poorer districts, as the comparatively light roads that these districts are able to finance are destroyed faster than they can be built. Rigid traffic regulation on a rational basis, depending on the local conditions, is a more positive help in local road improvement programs then even quite generous state-

aid moneys. Where it is impossible to finance roads that will economically handle heavy trucking, it is obvious that the only solution requires the elimination of such units during the first stages of improvement. Even the first stages of improvement, however, should probably permit the use of 2½-ton trucks.

6. The reasonable distribution of the tax burden and sound methods of raising construction and maintenance funds are fundamentals in the success of any program.

7. The classification of roads tends to stabilize procedure through successive administrations and tends to encourage a rational distribution of funds and well-balanced engineering type selection.

8. Sound general policy in the matter of handling growth in traffic volume apparently favors the principle of ultimately distributing traffic over moderate-priced parallel routes rather than by concentration on a few extremely wide and extremely expensive highways.

9. Sound general policy in the matter of pavement type seems usually to lead to the use of flexible pavements for the earlier stages of improvement programs, gradually strengthening the foundation and improving the character of the surface up to practically any standard of convenience, except for a very small percentage of the mileage (special-service roads), on which the outset there is no doubt that the rigid type of pavement is the most economical investment.

10. The success of any program depends on the ability of the executive to develop an efficient design, construction, and maintenance organization. In highway work as in any other line the whole matter hinges on personnel, and this particular phase of the problem can be well handled only under a liberal policy of public service salaries and recognition of ability.

The following chapters consider the details of engineering design.

CHAPTER II

LOCATION—GRADES AND ALIGNMENT

Location, grades, and alignment are the most permanent elements of a highway improvement and are entitled to reasonable forethought and to as much money as required to meet future engineering requirements suitable for the final stages of improvement of the road in question.

LOCATION

Selection of Route.—The selection of route depends on the purpose of the road, the topography between controlling points, and the stage of development of the community. Each case is a special problem, but there are certain fundamental facts worth considering. The basis of decision on general routes rests on good common sense and is not entirely an engineering problem. The road must go where it will do the most good, and it is up to the engineer to locate it in detail along the general route. The route location rests on reasonable answers to questions of the following nature: Where will the road do the most good to develop the natural resources of pioneer districts, or how can this route be located to serve the greatest number of people in well-settled communities, or how can this scenic road be built to give the most pleasure? If an attempt is made to solve all these problems strictly on the basis of the shortest distance and the easiest grades between terminals, the engineer would be in hot water. Any satisfactory solution considers the broad engineering principles of short distances, reasonable grades, and the smallest amount of rise and fall, but the final decision does not always rest on close analytical ton-mile cost hauling figures. To illustrate: recreational roads through national and state parks or forests are usually laid out to afford the most pleasure; grades and distance are sacrificed to obtain vistas, bold outlooks, and to reach points of historical interest or summer resorts. The cost of operating a car on such roads has no bearing on its usefulness, and a location based on a close analytical ton-mile hauling cost would be merely ridiculous. Consider a national highway from New York to San Francisco. Some through touring will occur, but its volume is very light and the cost of additional distance is not of any consequence to this class of traffic. More touring will go one-half or one-quarter of the way, but even this is of no great factor in comparison with short-distance traffic on the route. To lay out any long route on the basis of the shortest distance and easiest grades between terminals for through traffic and to disregard passing through or close to the most cities and villages on the route are

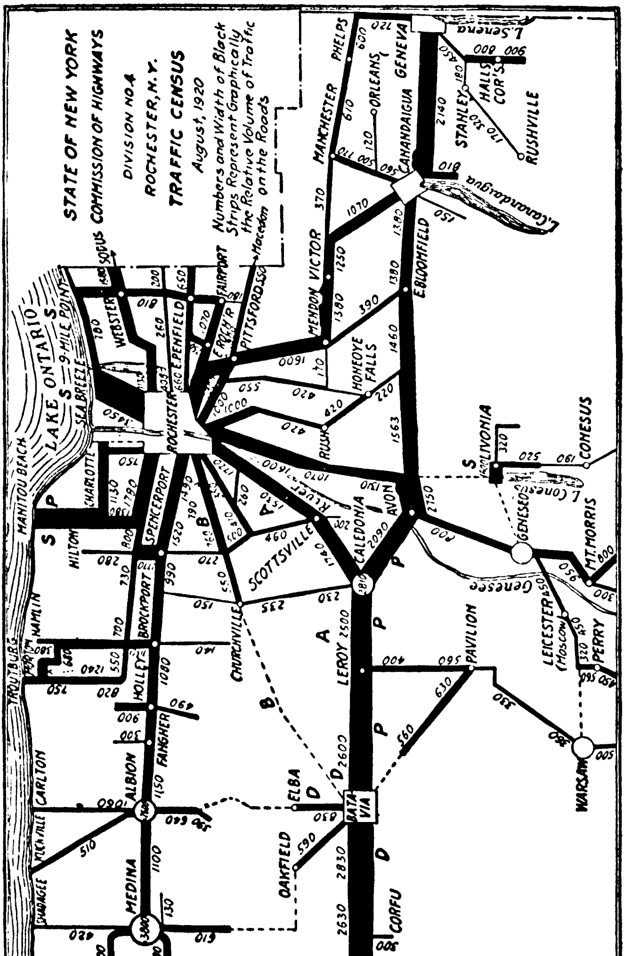


FIG. 10.—Traffic map. State Road System Division No. 4 Western New York.

evidently poor policies. This illustration is exaggerated to bring out the principle of route selection in well-settled communities, which is, namely: To pass through the most populous areas, and either close to or through the most cities and villages that can be consistently done without too much additional distance. This same principle applies to state and to purely local roads and may be summed up as direct contact with the greatest number of people.

As the distance between controlling points becomes less the factor of commercial hauling has a larger bearing on the selection of route until a point is reached where the engineering requirements of location govern the selection. That is, a reasonably low ton-mile hauling cost governs the short integral parts of any long-route location. At the present time (1926), motor freight hauling in competition with railroads is rarely economical for a distance of over 80 to 100 miles between terminals. This limit will probably fluctuate, but it is not likely to increase much, and for the time being it does not seem desirable to permit the factor of long-distance motor freight hauling to influence the selection of routes between large cities directly connected by rail over 100 miles apart. Where large cities are located closer than this, and there is a large volume of heavy motor hauling, it is possibly better to save distance by omitting some of the local service. Where large cities are isolated, heavy trucks rarely operate to outlying towns farther than 30 to 40 miles. Take a definite instance to illustrate this principle (see Fig. 10, p. 75). Rochester, N. Y., a city of about 280,000 population, is located 80 miles from Buffalo, a city of approximately 400,000 people. The first state route completed between these cities is shown in Fig. 10 as far as Batavia, a city of 14,000 people, and is designated on this map as Route A. This route was laid out in conjunction with state Route 6, the main east-and-west route, on the principle of local service, and it has served very satisfactorily for through traffic also. From the standpoint of through traffic between Rochester and Buffalo, the route marked B on the map (Fig. 10) is the logical route and this will undoubtedly be built in the near future.[1] That is, experience indicates that it is better to care first for the local service and then in the future, as traffic requires it, build new routes or partially relocate old routes for the further advantage of long-distance travel. A comparison of these two routes between Rochester and Batavia follows and shows the distinct advantage of Route B from the standpoint of through travel and of Route A for local service.

	Route A	Route B
Length, miles	37	31
Total rise and fall, feet	1850	1400
Number of railway shipping points served	15	9
Total railroad crossing	13	4
Railway grade crossings	9	1
Overhead or subway railway crossings	4	3

To give an idea of the traffic on this route in 1919 the following census (average 10-hr. count in summer season) is shown at different points:

[1] Route B completed in 1926.

ROUTE A (TRAFFIC CENSUS)

	Horse traffic		Motors		Total
	1 horse	2 horses	Cars	Trucks	
Between Rochester and Scottsville	30	30	850	100	1010
Between Scottsville and Caledonia	70	40	700	110	920
Between Caledonia and Le Roy	85	65	1600	180	1930
Between Le Roy and Batavia	15	10	1200	85	1310

As an additional point of interest, the new proposed through Route B from Bergen to Batavia fails to pass through two small settlements Byron and South Byron, because it would be necessary to use 2 miles extra distance for this local service—that is, Route B primarily considers through service. These villages can be served by a stub line. It is often desirable to by-pass villages and even certain cities on through routes on account of traffic congestion and the annoyance and danger of a large volume of high-speed traffic in the communities in question. These places can be served by stub lines or supplementary loops. As a rule, villages desire to have the main roads pass directly through them on account of the state aid for their street paving and the additional business derived from traffic, but the last feature does not amount to much unless they happen to be so located on the line that the traffic would naturally stop for some reason.

Pioneer Location.—To give an idea of the factors entering into the selection of routes in mountain districts, an example will be cited from Colorado, namely, the Bardine-Redstone road through the Sopris National Forest. This road was selected for improvement and advanced in order of construction by the U. S. Forest Service for the following reasons: An examination of the map will show that by a short road about 30 miles long over McClure's Pass the Carbondale and Paonia valleys can be directly connected. Without this road, it took a day's travel by rail to get from Carbondale to Somerset. The second reason for the road was to open up a promising farming section along the upper Blackwater, which had heretofore been confined to a cattle and sheep range on account of the impossibility of getting produce to the railroads. By the construction of moderatlely low-cost natural-soil road on a 5 or 6% grade over the low pass 3000′ above the valleys, intercommunication and new territory could be developed and a day's time in travel saved between two flourishing sections.

The foregoing discussion indicates, in a general way, some of the factors governing route selection.

Engineering Location.—A good detail location along the required general route results in the most effective road for the traffic that can be obtained for the available funds. Desires for perfection are controlled by the limitations of the community pocketbook. It is obviously desirable to obtain short distance, easy alignment, reasonable grades and to avoid locations which call for extremely expensive construction, such as rock work, flood, areas, etc. It is obviously desirable to avoid locations where snow drifts badly or fails to melt promptly in the spring, as the number of days in a year that a road is open has a large effect on its usefulness. It is obviously desirable to avoid needless railroad grade crossings. A summary of the engineering principles of location is given on page 81.

Extreme Refinements Impracticable.—An economic engineering location for commercial roads might be theoretically developed on the lowest ton-mile hauling cost to traffic. Practically, it is not yet reasonable to do this in many cases for the ordinary highway, and the reasons for disregarding this factor as the deciding element seem sound. Railroads have spent large sums to reduce the ton-mile cost and in their location the engineers make extremely careful comparative estimates of construction cost against operating cost. They consider the elements of shorter distance, curvature, light and heavy grades, etc. Many railroad engineers wonder why these considerations are not given more weight on highway work, considering the increase in mechanical transport. One of the evident reasons is that railroads get a direct tangible money return in dividends for their expenditure, and the return to the community on a public-road investment is too intangible. As a matter of interest, however, Tables 5 and 6 (p. 12) give approximate relation to distance, rise, and time as it affects operating costs. These data have been used by the author for some time as a basis for judgment in the comparison of lines.

It is undoubtedly true that, to get the full value of an improved road system, the engineering location must be made for the most efficient use of motor transport, but at the present time there is no possibility of obtaining or any justification for spending extremely large sums to reduce the hauling cost below that obtained by the usual modern highway design. If truck owners provided unlimited funds, a careful analysis would be justified on special commercial roads, but the following facts must be considered: the location of roads in well-settled districts are practically confined to existing rights of way except for minor relocations to avoid extreme grades or for safety reasons. These rights of way were not necessarily laid out with any regard to economic road location and, in fact, were often arbitrarily fixed by land section lines or locations where a poor road could be constructed in the past without much labor or cost. The cost of new rights of way for entire new locations and the difficulties of acquiring are prohibitive at this stage of development in road building except for unusual cases. The improved roads of today are only a progressive stage in the development of highway transport. The demand for them and the satisfaction in their use lie mainly in the fact that they provide a firm surface which can be used the year round, that they materially cheapen the cost of hauling, and that they make the use of light automobiles feasible for long and fast trips. The community is willing to pay a certain amount for the improvement in road conditions which the usual practice in modern road construction gives, but it is not willing to pay large additional sums for further reduction in ton-mile hauling costs. In the first place, only a comparatively few men would get a direct benefit from such expenditure. The indirect return to the community is too intangible. Much of the road traffic is pleasure traffic, and a few more gallons of gas mean nothing. If the owner did not spend his surplus for gas, he would spend it for ice-cream soda or the movies. There seems to be no way of making the few road users who would benefit by a further reduction in hauling cost pay the price of the necessary construction. It may be that for certain toll roads some time in the future or for exceptional cases in metropolitan districts a ton-mile cost location analysis could be used, but as yet this standard is too high for the usual road.

This does not mean that the engineer should not make an effort to get the best possible location that he can, but he should bear in mind that the first principle of general policy considering a comprehensive road system is mileage service, and aim to keep cost to a reasonable minimum in order to get the greatest mileage of road that will serve the purposes of the great majority of road users. For all roads except special-service commercial roads probably 90% of the traffic does not demand nor would it be particularly benefited by excessive refinements. Poor grade or alignment should never be used on high-class roads, as they are the fundamental features of the improvement and the only permanent features of construction. Liberal expenditures are justified, but there is a limit to expenditures for refinements that reduce mechanical operating costs to a minimum. The detail analysis of grade, alignment, section etc. given in Chap. II and III are intended to bring out the requirements of road design that are necessary for the satisfaction, safety, comfort, and comparative cheap hauling requirements of the average road users. These are the fundamentals which must be provided. Additional refinements beyond the fundamental requirements are desirable if the funds are available from the proper sources. By the "proper sources" is meant the actual road users benefited by the additional cost of construction.

Saving in distance is valuable; saving in total rise is valuable; easy grades and the elimination of sharp curves are desirable. Every effort is made to accomplish these results, utilizing the existing roads where it is necessary, making minor relocations to avoid extreme grade or danger because the sentiment of the community approves these measures, but always bearing in mind that today and for a long time to come mileage is the prime requisite of programs. It is possible in the sparsely settled communities to make better engineering locations, as far as right-of-way handicap is concerned, but in these districts shortage of funds often plays havoc with intentions.

Value of Saving Distance and Rise.—It is well to bear in mind what distance saving is worth and what a saving in total rise is worth. The data given are, of course, of only general value, as the fluctuating cost of motor operation, the types of hauling, and special conditions of all sorts affect the figures. They, however show in a general way that it is well worth while to reduce traffic losses arising from these elements of needlessly poor location or design.

A. R. Hirst gives the following conservative figures on the value of saving distance:

"If the very conservative sum of 10 cts. per mile is allowed for each mile of travel saved, the saving of a mile in distance on highways carrying the following average number of vehicles per day will save the traveling public the given amount per year, which is the interest at 5% on the amount given in the third column."

TABLE 17.—VALUE OF A MILE IN HIGHWAY DISTANCE SAVED

Average number of vehicles per day	Saving to owners per year	Saving capitalized at 5% equals
100	$ 3,650	$ 73,000
250	9,125	182,500
500	18,250	365,000
750	27,375	547,500
1,000	36,500	730,000
2,000	73,000	1,460,000
5,000	182,500	3,650,000
10,000	365,000	7,300,000

The value of eliminating rise cannot be figured with any degree of accuracy, as there are too many indeterminate and variable factors, but in the authors' opinion it is not likely that the capitalized value of saving in yearly operation due to eliminating 1′ of rise and fall per 100 vehicles per day on long routes will exceed $60 on light grades or $400 on heavy grades. For small grading reductions on short hills the time factor is of no consequence and the practical value of saving a foot rise and fall is not probably more than one-third of these figures.

It is very evident that considerable expenditure is justified to reduce distance and rise, but it is also evident that it would be impracticable to carry this method of location to its logical conclusion by expenditures in any way approximating the figures given. That is, the location of a free public road financed by a general tax with no direct revenue return can hardly be analyzed from the same point of view as a trunk-line railroad.

Relocations of Existing Highways.—Everyone would prefer to have scientifically located highways. A great many engineers believe that the time has come to make extensive relocations. It is self-evident that relocations which reduce the construction cost of the proposed road as well as reduce motor operation costs should be made at once. It is surprising how often even such relocations are not made, and it is desirable to impress on the men in charge of surveys that they should continually bear in mind the necessity of such relocations and not feel that they must follow the present road lines where these conditions prevail. There seems to be no question that expenditures for relocations necessary to obtain reasonably good grades and alignment are justified at the present stage of road programs, but it is believed that extensive relocations involving excessive refinements must be gradually worked out except for a few extreme cases and that practically it will be easier to accomplish and fairer to the general public to do most of this work under reconstruction programs financed by direct vehicle taxes rather than to attempt it at this time in the first stage of improvement.[1]

In case a relocation is necessary, no halfway measures should be allowed. In too many cases even on fairly important state roads in rich communities relocations have been made on the basis of 9% grades when it was perfectly possible to get 7% or less. Halfway treatments of this kind are worse than nothing.

To illustrate present practice on relocations, the following quotation from the Iowa Highway Department "Field Manual" is given. The limiting grade of 6% mentioned does not agree with the recommendation given on page 105, but the general scope of the data strengthens the discussion at this point.

Relocation.[2]—"Where the topography is flat or gently rolling the profiles readily lend themselves to satisfactory grades at a moderate cost, and relocations to any extent are seldom necessary. But in the rougher country, relocations will frequently be necessary and the field man must constantly watch for opportunities to better the alignment, avoid steep hills, or improve stream crossings by relocations. The necessity for or advisability of relocating must always be balanced against the cost, and, in general, it is true that a proposed change of any magnitude is advisable only when it can be shown that such change will be economical or will produce a decidedly better road. It is therefore important that the cost of relocation be thoroughly investigated. In this connection the field man must remember to take into consideration the various improvements along the existing road, such as farm buildings, orchards, permanent bridges and culverts, heavy cuts and fills, etc.

"The following instructions should be followed:

"*a.* In all cases where it appears that an excessive amount of earthwork will be required to reduce the present road to 6% grades, the possibility of relocations to reduce grades to 6% or less shall be fully investigated.

"*b.* In cases where there is a succession of grades which may be reduced to 5 or 6%, but which cannot be reduced below that figure without considerable work, the question of relocation should be fully investigated.

"*c.* In case of doubt as to the feasibility of any relocation, a survey should always be made.

"*d.* In all cases where relocations are surveyed a survey shall be made on the old road also.

"*e.* In the case of minor relocations the margins of the old roadway should always be shown by a sketch indicating the old roadway by dotted lines, and by data in the cross-section notes. In such cases the survey of the old road may consist only of extending the cross-sections over the same.

"*f.* The notes shall show which location is to be used or shall state that the determination of which route to follow cannot be made until the notes are worked up in the office. The chief of party shall enter this notation in the field notes after consultation with the district engineer."

Summarized Principles of Location.—Climatic, drainage, and soil conditions govern a location in respect to avoiding bad snow conditions, flood areas, needless stream crossings, needless railroad grade crossings, slide or swamp formations, and excessive rock work. The general requirements of line and grade discussed in this chapter are summarized as follows: the various principles are repeated conversely under the headings of Grade, Alignment, Distance, Rise and Fall, and Cut and Fill Grade Reductions.

[1] See pp. 12, 13 for an approximate basis of comparing the value of alternate location.

[2] "Field Manual," Iowa Highway Dept.

curve connections. Grade-line design in connection with alignment considers the relative values of distance against rise and fall and the relative safety of light grades with curved alignment as compared with steep grades and straight alignment.

The effect of grade may be roughly summarized as follows:

An increase in the rate of grade decreases the load that can be hauled up the grade by a fixed power.

An increase in the rate of grade increases the expenditure of energy to maintain a fixed speed climbing the grade.

An increase in the rate of grade decreases the speed for a fixed power.

An increase in the rate of grade increases the wear and tear on mechanical outfits.

The mechanical energy expended in climbing is partially balanced by the reduction of energy expended on down grades.

The mechanical energy expended in climbing affords a very definite basis of comparison of the value of the travel in one direction. The expenditure of energy on down grades is indefinite, and while it affects the total operating cost on a grade, it cannot be given as much weight in the conclusion as the first method. The effect of grade on the depreciation and repair of mechanical equipment is indefinite, but it is certain that it bears some relation to the rate of grade.

Grade selection depends on considerations of safety, convenience, traffic operating cost, and the cost of construction and maintenance. Cost of traffic operation is not always the most important factor. It must often give way to considerations of safety or initial construction cost.

Reasonably low rates are desirable. The whole question of grades lies in the decision of what is reasonable for a specific case. A summary of practical rules for location and cut and fill grade line design is given on page 81.

Maximum Grades.—The subject of maximum grades will be considered from the following standpoints:

1. Relative importance of horse and automobile traffic in the selection of grade.
2. Effect of grade on horse traffic.
3. Effect of grade on motor traffic.
4. Current practice in maximum grades.
5. Practical considerations governing the selection of grade.
6. Effect of ruling grade on cost.
7. Recommended general practice.

Relative Importance of Horse and Auto Traffic in the Selection of Maximum Grade.—Tables 18 and 18A show the rapid growth of motor traffic on the main roads of Massachusetts and the general character of the traffic on secondary roads in western New York.

Everyone is familiar with this change in the character of highway traffic. Maximum grades have a radically different effect on horse and single-unit motor traffic, and it is necessary to come to some reasonable conclusion as to which kind of travel should govern the design.

TABLE 18

	1912	1915	1918	Per cent of increase, 6 years
Automobiles and trucks....	50,132	102,633	191,019	280
Motorcycles..	5,034	9,520	12,708	150
Operators and chauffeurs..	65,600	133,700	225,272	240
Motor vehicle fees......	$616,236	$1,235,723	$2,159,257	250

AVERAGE DAILY TRAFFIC ON MAIN ROADS IN MASSACHUSETTS

	1909	1912	1915	1918	Per cent of increase, 9 years
Light horse..................	91	68	40	24	−73½
Heavy horse.	88	88	72	43	−51
Total horse	179	156	112	67	−62½
Automobiles and light trucks...	131	280	555	923	+604
Heavy trucks..............		17	45	75	+341[a]
Total motors...........	131	297	600	998	+661
Total vehicles..	310	453	712	1065	+243

[a] In 6 years.

PER CENT OF TOTAL TRAFFIC

	1909	1912	1915	1918
Light horse...	29	15	5½	2
Heavy horse.....	28	19	10	4
Total horse.....	57	34	15½	6
Trucks.......		4	6½	7
Motors..	43	62	78	87

TABLE 18A.—DAILY TRAFFIC COUNTS ON SELECTED "LOCAL-SERVICE" STATE COUNTY ROADS IN WESTERN NEW YORK

Number of roads	Number of miles	Horse traffic			Motor traffic		
		1 horse	2 horses	Total horse	Light cars	Trucks	Total motors
14	60	45	39	84	250	36	286

Horse traffic, per cent of total........................ 23
Motor traffic, per cent of total. 77

NOTE.—On the main state route roads the percentage of horse traffic corresponds very closely with the Massachusetts results given in Table 18.

For main roads in well-settled districts, single-unit motor traffic may well control maximum-grade design. For local service roads in agricultural districts or in mountain pioneer regions, horse traffic requirements should control maximum-grade line design.

Maximum Grades from the Standpoint of Horse Traffic. *Difficulty of Ascent and Safety of Descent.*—The factors controlling ease and safety of ascent and descent have different values for different surfaces, but as most of the roads will in time be hard surfaced and as all parts of the design should fit into the final improvement, this part of the grade argument is made primarily for hard-surfaced conditions.

European observers claim that on a stone road 5% is the maximum grade that can be descended safely by a trotting team without brakes, and that 12% is the maximum that can be safely descended with brakes. By the use of the sliding shoe or locked wheels, freighters in the Rockies descend 20% grades without much difficulty on ordinary natural-soil roads. Safe descent with brakes need not be considered except in rare cases, as it would result in a grade far beyond ordinary practice. Safe and easy descent without brakes is more important for light rigs than for heavy hauling, but as this class of traffic has been practically eliminated by cheap automobiles it need not be given much weight. Descent, therefore, plays only a minor part in grade selection except where the alignment is bad.

Hauling Power.—The writers know of no careful records of actual maximum loads that can be hauled up different hard-surfaced grades by an ordinary team; it is probably better to discuss this point theoretically, as any experiments would be affected by too many variable local conditions to be worth much as a basis of comparison. As a check on the theoretical discussion, records of loads on extreme mountain grades are given on page 91, which show that, for all practical purposes, Table 24 of theoretical loads is fairly close and is on the safe side.

A summary of Prof. I. O. Baker's discussion of maximum team loads is given below, and through his courtesy it is possible to include a collection of tables taken from his work, "Roads and Pavements."

Various trials have determined that the normal tractive power of a horse traveling 3 m.p.h. for 10 hr. a day is approximately one-tenth of its weight; that when hauling up a steep grade it can exert one-fourth of its weight for a short time; that for, a continuous exertion of one-fourth, the grade should not be over 1200′ long and if over that resting places should be provided every 600 to 800′; that in starting and for a distance of 50 to 100′, one-half of its weight can be used; and that the net tractive power ordinarily exerted by a horse on a grade equals one-fourth its weight—the effort required to lift itself, or approximately $0.25W - W \times$ per cent of grade expressed in hundredths, *i.e.*, $0.25W - 0.04W$ for a 4% grade. This undoubtedly gives a reasonable basis for ordinary hauling conditions, but from data obtained by the author in connection with freight hauling in mountain regions it is evident that a good draft horse will exert more than $0.25W$ on moderately short, sharp pitches of a long climb if allowed to rest at intervals of 200 to 300′. The evidence indicates that a value of $0.35W$ is about right for such conditions.

Table 19 shows the effective power developed by an ordinary team of 1200-lb. horses with moderate exertion and Table 19*A* the power of a first-class team of 1600-lb. horses exerting their full strength.

TABLE 19.—ORDINARY STOCK MODERATE EXERTION

Grade, %	Theoretical net tractive effort	Tractive effort, in pounds
Level	$0.10W$	240
2½	$0.25W - PW$	540
4	$0.25W - PW$	504
5	$0.25W - PW$	480
6	$0.25W - PW$	456
7	$0.25W - PW$	432
8	$0.25W - PW$	408
9	$0.25W - PW$	384
10	$0.25W - PW$	360

W = weight of team, 2400 lb.
P = per cent of grade in hundredths.

TABLE 19*A*.—DRAFT STOCK FULL POWER

Grade, %	Theoretical net tractive effort	Tractive efforts, in pounds
5	$0.35W - PW$	960
6	$0.35W - PW$	928
7	$0.35W - PW$	896
8	$0.35W - PW$	864
10	$0.35W - PW$	800
12	$0.35W - PW$	736
14	$0.35W - PW$	672
16	$0.35W - PW$	608
18	$0.35W - PW$	544
20	$0.35W - PW$	480
22	$0.35W - PW$	416

W = weight of team, 3200 lb.
P = per cent of grade in hundredths.

Grade and Rolling Resistance.—This power is used in overcoming axle friction, gravity resistance, and rolling resistance.

The axle friction is small, amounting to 3 or 4 lb. per ton for American farm wagons.

Grade resistance (gravity) equals load × per cent of grade expressed in hundredths, and, expressed in pounds per ton of load, equals $2000 \times P$.

The rolling resistance varies for different surfaces and for each surface depends on the diameter of wheel, width of tire, speed of travel, and the presence or absence of springs on the wagon. The best diameter of wheels, best width of tires, and the use of springs as they affect the ease of hauling for both farm and road use are problems for the wagon manufacturers.

Morin, a French engineer, concluded from a series of careful experiments that the harder the surface of the road the less effect width of tire had on rolling resistance. Comparatively hard surfacing and small differences in wheel diameter are taken for granted here, and these factors can be disregarded. As a matter of interest, Tables 20 and 22 are included to show the results of experiments on different soils and roads.

The question of wide tires affects road design chiefly in connection with the distribution of load over a safe area and will be taken up under Macadam Design.

Table 23 gives the average rolling resistance in pounds per ton of load on different pavements for the ordinary farm wagon driven at ordinary speeds.

TABLE 20.—EFFECT OF WIDTH OF TIRE UPON TRACTIVE POWER[1]
RESISTANCES IN POUNDS PER TON

Ref. No.	Description of the Road Surface	Diameters of the Front & Rear Wheels respectively									
		3'-6" & 3'-10"		3'-6" & 3'-10"		3'-8" & 4'-6"		3'-6" & 3'-10"		3'-8" & 4'-6"	
		Width of Tires									
		1½"	4"	1½"	4"	1½"	4"	1½"	3"	1½"	3"
1	Sod							283	239	189	228
2	Earth road (hard)		108					152	152	114	114
3	" " (muddy)		243	268	304	236	254			265	228
4	Sand " (hard)	199	162	171	164	141	168				
5	" (deep)	371	351								
6	Gravel road (good)		*	98	117	83	80			66	76
7	Wood Block (round)	51	49	61	70	35	46		54	28	38

[1]Pamphlet by Studebaker Brothers Manufacturing Company, 1892.

TABLE 21.—EFFECT OF SIZE OF WHEELS ON TRACTIVE RESISTANCE[1]
POUNDS PER TON

Ref. No.	Description of Road Surface	Mean Diameter of Front & Rear Wheels		
		50"	38"	26"
1	Macadam, slightly worn, fair condition	57	61	70
2	Gravel road, sand 1" deep, loose stones	84	90	110
3	" " upgrade 2.2%, one-half inch wet sand, frozen below	123	132	173
4	Earth road. Dry and hard	69	75	79
5	" " ¾" sticky mud, frozen below	101	119	139
6	Timothy & blue grass sod, dry grass cut	132	145	179
7	" " " " " wet & spongy	173	203	281
8	Cornfield; flat culture across rows, dry	178	201	265
9	Plowed ground; not harrowed, dry & cloddy	252	303	374
10	Average Value of Tractive Power	130	148	186

Experiments of T. I. Mairs at the Missouri Agricultural Experiment Station.

TABLE 22.—TRACTIVE RESISTANCE OF BROAD AND NARROW TIRES.[1] RESISTANCE IN POUNDS PER TON

Ref. No.	Description of Road Surface	Width of Tire		No. of Trials
		1½"	6"	
1	Broken Stone, Road; hard, smooth, no dust, no loose stone	121	98	2
2	Gravel Road; hard and smooth; a few loose stones	182	134	2
3	" " " no ruts, large quantity of sand	239	157	1
4	" " new gravel, not compact, dry	330	260	1
5	" " wet, loose sand 1" to 2½" deep	246	254	2
6	Earth Roads. Loam, dry, loose dust 2" to 3" deep	90	106	2
7	" " " dry and hard, no dust, no ruts, nearly level	149	109	3
8	" " " stiff mud, drying on top, spongy below	497	307	1
9	" " " mud 2½" deep, firm below	251	325	1
10	" " Clay, sloppy mud, 3" to 4" deep, hard below	286	406	1
11	" " " dry on top but spongy below	472	422	2
12	" " " dry on top but spongy below	618	464	5
13	" " " stiff deep mud	825	551	1
14	Mowing Land. Timothy sod, dry, firm, and smooth	317	229	1
15	" " " " moist	421	305	1
16	" " " " soft and spongy	569	327	1
17	Pasture " Blue grass sod, dry, firm, and smooth	218	156	2
18	" " " " " soft	420	273	2
19	" " " " " soft	578	436	1
20	Stubble " Corn stubble, no weeds, dry enough to plow	631	418	2
21	" " " " some weeds, dry enough to plow	423	362	1
22	" " " " in Autumn, dry and firm	404	256	2
23	Plowed " Freshly plowed, not harrowed, surface rough	510	283	1
24	" " " " harrowed, smooth, compact	466	323	1

[1] Missouri Agricultural Experiment Station Bulletin No. 39.

TABLE 23[1]

Kind of pavement	Rolling resistance, in pounds per ton of load
Asphalt...... ..	30– 70
Brick or concrete	15– 40
Cobblestones.	50–100
Earth roads..	50–200
Gravel roads	50–100
Macadam roads	20–100
Plank.......	30– 50
Stone block.	30– 80
Wood block	30– 50

[1] Baker's "Roads and Pavements."

Effect of Grade on Loads.—For a comparative estimate take a value of 40 lb. per ton of load, including axle friction, on macadams and rigid pavements and 100 lb. per ton for earth roads in fair shape. The resistance to the effective tractive power of the team per ton of load is therefore $40 + (2000 \times P)$ on the hard-surfaced roads, and $100 + (2000 \times P)$ for earth roads, and the maximum load expressed in tons for any grade equals

$$\frac{\text{Effective tractive power of team for that grade}}{\text{Resistance per ton of load for that grade}}.$$

Using the tractive powers of the ordinary team shown in Table 19, Table 24 is constructed. It is chiefly useful for a comparison of the effect of grade on load, but all evidence indicates that the loads given correspond closely to practice. Table 24A shows loads for extreme team exertion as compiled in Table 19A. The loads given include weight of wagon.

TABLE 24.—ORDINARY STOCK MODERATE EXERTION

Grade, per cent	Effective tractive effort, pounds	Improved roads		Earth roads	
		Resistance in pounds per ton of load	Maximum load, in tons	Resistance, in pounds per ton	Maximum load, in tons
Level	240	40	6.0	100	2.4
2½	540	90	6.0	150	3.6
4	504	120	4.2	180	2.8
5	480	140	3.4	200	2.4
6	456	160	2.9	220	2.1
7	432	180	2.4	240	1.8
8	408	200	2.0	260	1.6
9	384	220	1.7	280	1.4
10	360	240	1.5	300	1.2

TABLE 24*A*.—DRAFT STOCK EXTREME EXERTION

Grade, per cent	Effective tractive effort, pounds	Hard-surfaced roads: Resistance, in pounds per ton	Hard-surfaced roads: Maximum load, in tons	Earth roads: Resistance, in pounds per ton	Earth roads: Maximum load, in tons
5	960	140	6.8	200	4.8
6	928	160	5.8	220	4.2
7	896	180	5.0	240	3.7
8	864	200	4.3	260	3.3
10	800	240	3.3	300	2.7
12	736	280	3.0	340	2.2
14	672			380	1.6
16	608			420	1.4
18	544			460	1.2
20	480			500	1.0
22	416			540	0.8

Effect of Length of Grade on Maximum Load.—In mountain-road design where a long ruling grade is used it is often economical to introduce short stretches of steeper grade to avoid extremely expensive construction and to improve alignment. In order to determine the maximum short grade (not exceeding 300 ft. in length) that can be used in connection with a long ruling grade without reducing the team load, Table 24*B* has been compiled for a 2400-lb. team.

TABLE 24*B*.—EQUIVALENT LONG AND SHORT GRADES FOR HARD-SURFACED CONDITIONS

Long ruling grades tractive effort 0.25*W*, 2400-lb. team.		Short maximum grade tractive effort 0.35*W*, 2400-lb. team.	
Grade, %	Maximum load, tons	Grade, %	Maximum load tons
5	3.4	7	3.7
6	2.9	9	2.8
7	2.4	10	2.5
8	2.0	12[a]	2.0

[a] Twelve per cent is the practical limit (on account of safe descent) on any road of sufficient importance to be considered from an engineering standpoint.

This principle can also be applied to a long cut and fill grade reduction with a very material saving in cost, but if used the steeper rate should not be over 250 to 300′ long and should be at the bottom of the hill.

Records of Team Loads.—H. G. McPheters and F. F. Roberts have compiled the following data on team freighting in the Rocky Mountain region. It is practical data obtained from personal experience and strengthens the force of the theoretical discussion. The loads given are net and do not include wagon weights. They represent usual freighting loads which are practical maxima.

HEBER FRUITLAND ROAD, STATE OF UTAH

Daniels Canyon Section

Earth road in fair shape.
Long 8% grades.
Short 15% grades.
Net load for four-horse team, 3500 lb. (during summer).

GALENA SUMMIT ROAD, STATE OF IDAHO

Natural-soil road in fair shape.
Maximum grade (Salmon River side), 20%.
Maximum grade (Wood River side), 17%.
Load for one team, 1800 lb. (during summer).
Load for two teams, 4000 lb. (during summer).
Load for three teams (six horses and two wagons loaded 5000 lb. on lead wagon and 4000 lb. on trail taking one wagon at a trip up the mountain).

TRAIL CREEK SUMMIT ROAD, STATE OF IDAHO

Natural-soil road (fair condition during summer).
Maximum grade, 22%.
Load for one team, 1200 lb.
Load for two teams, 2500 lb.
When freighting by teams was the principal mode of transportation, there were used on this road several outfits of 24 mules hooked to four wagons loaded as follows: lead, 14,000 lb.; lead swing, 10,000 lb.; swing, 8000 lb.; and trail, 4000 lb. Two men handled the whole outfit, which was certainly a man's job.

ROCKY BAR ATLANTA ROAD OVER BALD MOUNTAIN

Natural soil.
Maximum grade, 16%.
Load for one team 2000 lb.
Load for two teams 4000 lb.
A large amount of freight is carried over this road by auto trucks at the present time.

The Theoretical Advantage of Certain Grades.—From Tables 24, 24*A*, and 24*B* and the previous discussion the grades that theoretically fulfil certain traffic requirements can be selected.

1. On hard-surfaced roads the same load that can be drawn up a 2½% grade by reasonable extra exertion of a team can be hauled on a level with ease. This makes a perfectly balanced design from the standpoint of team hauling. The theoretical load is 6 tons. For earth roads 5% fulfils this same condition with a theoretical team load of 2.4 tons.

2. Five per cent is the maximum grade that fulfils the requirement of safe descent at a trot without brakes. This is of little importance under modern traffic conditions.

3. The same load that can be hauled up a 7% hard-surfaced grade can be drawn on a level dirt road in fair condition; a 7% grade therefore does not reduce the load of a team which must travel over even a level earth road for part of the distance. The theoretical load is 2.4 tons.

4. The use of short maximum grades of greater rate than the long ruling grades does not reduce the maximum load, provided they are proportioned as follows for hard pavements and do not exceed 250 to 300′ in length:

Long	5%	Short	7%
Long	6%	Short	9%
Long	7%	Short	10%
Long	8%	Short	12%

5. Twelve per cent is the practical limit of grade for even unimportant roads on account of safe team descent with heavy loads.

As a matter of fact, the selection of grade depends more on the requirements of the traffic and the topography of the country than on these theoretical advantages.

Effect of Grade Selection on Motor Traffic.—Safety, convenience limitations of load, and cost of vehicle operation must be considered.

Safety of Motor Operation.—Motor accidents are quite frequent due to the sudden application of brakes on steep hills in rainy or sleety weather where the road has a hard, smooth surface. Accidents of this kind occur even on level grades, but are not frequent up to 5% rates of grade; beyond 5% they occur with rapidly increasing frequency and become a real cause of danger on grades over 7% even when the alignment is straight. From the standpoint of safety, 7% is a maximum rate for hard-surfaced roads.

Convenience of Motor Operation.—Drivers dislike to be forced into second or low gear. If it is possible to determine approximately the rate of grade at which most cars or trucks shift gear, this information has some bearing on grade selection. It is, of course, difficult to figure this closely as motor design improves, gear ratios vary, cars run on varying degrees of efficiency, gasoline varies in quality, etc., but as a matter of interest the authors' experience indicates that the average light pleasure car of the year 1919 shifts into second gear at about 7% and that very little gear shifting is necessary on long 6% grades. W. C. Slayton, a truck fleet manager, says that his 5-ton standard gear ratio trucks generally drop into second at about 5% and that very little shifting would be required on long 4% grades. Passenger autos drop into low at about 10% and the 5-ton trucks into low at about 8%.

From the standpoint of convenience in driving pleasure cars, these premises, if they apply, indicate that if for any reason a 6% grade cannot be obtained a 10% might just as well be used and that heavy expenditure to get a 7 or an 8% has no bearing on the convenience of the road. This applies only to scenic routes. In the same way for truck hauling, if a 4% cannot be obtained there is no object from the standpoint of convenience in using lesss than an 8%. Other factors, however, apply to reduce this extreme jump,

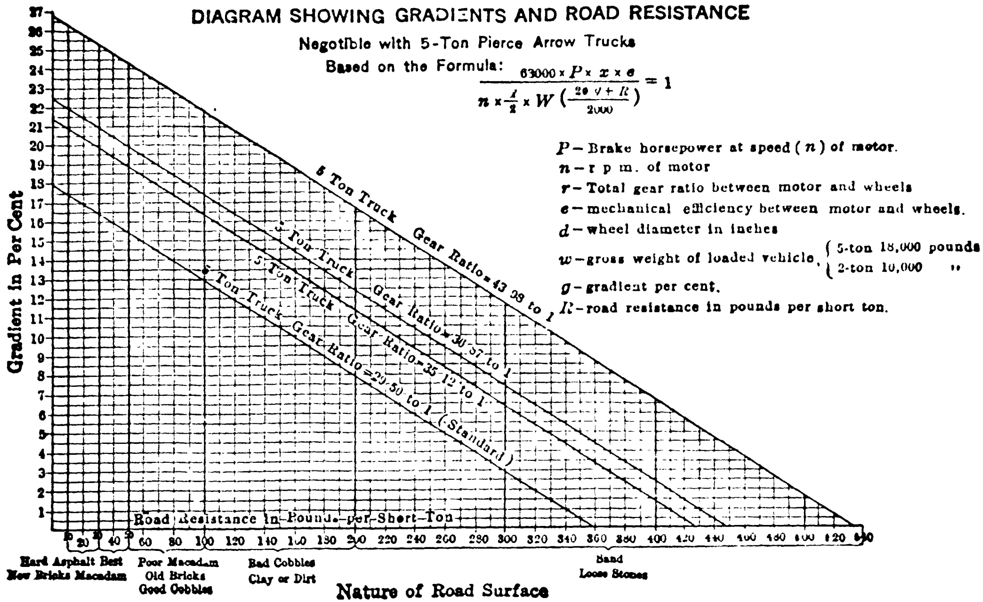


DIAGRAM SHOWING GRADIENTS AND ROAD RESISTANCE
Negotible with 5-Ton Pierce Arrow Trucks
Based on the Formula: $\frac{63000 \times P \times x \times e}{n \times \frac{d}{2} \times W \left(\frac{20\,g + R}{2000}\right)} = 1$
P — Brake horsepower at speed (n) of motor.
n — r p m. of motor
r — Total gear ratio between motor and wheels
e — mechanical efficiency between motor and wheels.
d — wheel diameter in inches
w — gross weight of loaded vehicle, 5-ton 18,000 pounds. 2-ton 10,000 "
g — gradient per cent.
R — road resistance in pounds per short ton.
5 Ton Truck Gear Ratio = 43.98 to 1
5 Ton Truck Gear Ratio = 39.37 to 1
5 Ton Truck Gear Ratio = 35.12 to 1
5 Ton Truck Gear Ratio = 29.50 to 1 (Standard)
Gradient in Per Cent
Road Resistance in Pounds per Short Ton
Hard Asphalt Best
New Bricks Macadam
Poor Macadam
Old Bricks
Good Cobbles
Bad Cobbles
Clay or Dirt
Sand
Loose Stones
Nature of Road Surface

CHART A.

OPTIONAL GEARING ON FIVE-TON MODEL

The first option is our standard gearing and will be supplied on all orders unless otherwise specified. This gearing should be used where the truck is to traverse good hard roads at all times, and where the grades do not exceed 10%.

The second option gives great pulling power on the low speeds, and the standard speed of 14 miles per hour on high gear. This gearing should be used only where the truck has to pull through a very short portion of poor road and the great majority of the running is done on direct drive. This option is popular with contractors, etc.

The third option is especially suited for districts where by nature of roads or traffic conditions a high speed is undesirable, or in hilly country, where the road surfaces are good. This gearing is standard equipment on the long wheel base model.

The fourth option should only be used where the road surfaces are exceedingly poor, and the country very hilly. We do not advise using this gearing except in extreme cases.

as discussed later. It should, however, be borne in mind that, if trucks are operating regularly over a stated route, special gear ratios can be and are used to meet the existing grades (see Chart A, note, p. 95). Gear shift, therefore, plays a minor part in grade selection.

Effect of Grade on Load.—Modern single-unit trucks or passenger cars have enough power to carry their full-weight load up any grade within the limits of standard highway design. Limitation of load has no effect on maximum grade design from the standpoint of single-unit motors.

Long trailer train loads are limited by maximum grades and demand low maximum grades. It should, however, be remembered that this type of haulage is slow, clutters up the highway, is dangerous to ordinary traffic, and only in rare instances for special roads should it be permitted or grades designed for such haulage.

Chart *A* shows the ability of Pierce Arrow trucks to pull on different kinds of roads and different grades.

Effect of Grades on Motor Operation Cost.—For a detailed discussion of this complicated subject the reader is referred to standard highway books and government reports on road economics.[1] The application of Tables 5 and 6 (p. 12) will give a good basis for the approximate solution of special cases desired and the following short discussion gives summarized general conclusions.

As a preamble to any discussion of the effect of motor operation cost on the design of grades, it is just as well to bear in mind that the American high-powered passenger car is not designed primarily for low fuel cost per ton mile; that the average owner does not attempt to reduce his fuel costs to a minimum by careful attention to mechanical adjustments; and that more gasoline and oil are wasted yearly through sheer individual carelessness and ignorance than could possibly be saved by the greatest refinements of design.

The cost of a business trip by motor is reckoned as the sum of the cost of operating the vehicle and the time consumed in making the trip. The value of small-time increments gained by grade reductions is so small that for practical purposes it can be ignored in most cases. The cost of motor operation includes gas, oil, tires, repairs, and depreciation. The effect of grades on gas and oil consumption can be figured with reasonable accuracy, and it has been found that tires, repairs, and depreciation are almost directly proportioned to gas consumption.

Table 25 gives the relation between rolling resistance and gas consumption.

Table 26 gives the approximate relative cost of different items of operation for different types of road vehicles. Table 4 (p. 11) gives the approximate total cost of operation per mile.

From these or similar data it is simple to approximate the effect of grade on the cost of operation while climbing the grade.

The cost of operation downhill is more uncertain.

Table 27 page 100 gives an approximation of the costs of operating up and down different grades.

[1] *Bull.* 69, Iowa State College. "Location, Grading and Drainage of Highways," McGraw-Hill Book Company, Inc. "The Construction of Roads and Pavements," McGraw-Hill Book Company, Inc.

TABLE 25.—AVERAGE ROLLING RESISTANCE AND RELATIVE FUEL CONDITIONS[1]

Type and condition of roadway surface	Average values rolling plus air resistance				Relative fuel consumption with that on surface for which R = 30 lb. per ton taken as unity			
	Solid tires, 10 m.p.h.	Pneumatic tires, 15 m.p.h.	Pneumatic tires, 25 m.p.h.	Pneumatic tires, 35 m.p.h.	Solid tires, 10 m.p.h.	Pneumatic tires, 15 m.p.h.	Pneumatic tires, 25 m.p.h.	Pneumatic tires, 35 m.p.h.
Portland cement concrete—best, newly finished	30	22	27	35	1.00	0.89	0.96	1.07
Portland cement concrete—rough due to poor work	36	30	35	42	1.08	1.00	1.07	1.16
Portland cement concrete—average good condition	32	27	32	39	1.02	0.96	1.02	1.12
Asphaltic concrete, coarse-graded type, average yearly temperature—best	30	35	30	37	1.00	0.93	1.00	1.09
Asphaltic concrete, coarse-graded type, average yearly temperature—average	33	27	32	39	1.04	0.96	1.02	1.12
Sheet asphalt at average yearly temperature—best	28	23	28	35	0.97	0.91	0.97	1.07
Sheet asphalt at average yearly temperature—average	36	30	35	42	1.08	1.00	1.07	1.16
Bituminous filled brick—average—no filler on surface	30	26	31	38	1.00	0.95	1.01	1.11
Grout-filled brick—average	37	30	38	45	1.09	1.00	1.11	1.20
Wood block bare of filler—average uniform surface	35	30	34	40	1.07	1.00	1.05	1.13
Gravel—best clay bound	40	35	40	47	1.13	1.07	1.13	1.23
Gravel fair to poor—rough spots, some loose material	55	50	55	62	1.33	1.27	1.33	1.42
Gravel poorest condition—rough and many loose pieces	60	55	60	65	1.40	1.33	1.40	1.47
Gravel, Iowa yearly—average, approximated	50	45	50	57	1.27	1.20	1.27	1.36
Natural soil—good, well graded, and patrol maintained	45	35	40	47	1.20	1.07	1.13	1.23
Natural soil—soft (or slightly "spongy")	70	70	75	80	1.54	1.53	1.60	1.67
Natural soil—Iowa yearly average, approximated	55	45	50	58	1.33	1.20	1.27	1.37
Snow—2" thick and well packed	55	50	70	..	1.33	1.27	1.53	
Snow—about 4" thick, slightly packed	75	70	..	..	1.60	1.53		
Snow—about 4" thick, slightly packed, chains on wheels	75	..	..	..	1.60			
Average for best paved surfaces—concrete, asphalt, brick and wood block	30	22	27	37	1.00	0.89	0.96	1.09
Average for partly worn pavements, *i.e.*, in fair average condition	35	30	35	42	1.07	1.00	1.07	1.16
Yearly average for best gravel of type used on trunk line	45	40	45	55	1.20	1.12	1.20	1.33
Yearly average for ordinary gravel found on secondary roads	55	50	55	65	1.33	1.27	1.33	1.47
Yearly average for second-class earth roads under good maintenance	65	60	63	75	1.47	1.40	1.44	1.60
Yearly average for best earth roads under heavy traffic and well maintained	55	50	53	65	1.33	1.27	1.31	1.47

[1] *Bull.* 69, Iowa State College.

As one-half of traffic is uphill and one-half is downhill on most roads, the final cost for any grade must be the average of these two values. Tables 5 and 6 (p. 12) were derived on this basis.

The detailed study of a large number of cases have led to the following general conclusions:

TABLE 25A.—RELATION BETWEEN FUEL CONSUMPTION AND TRACTIVE RESISTANCE[1]

Type	Source of data			Relative tractive resistance
	Ames	Canadian engineer	A. N. Johnson	
Good pavement	1.00	1.00	1.00	1.00
Macadam and bituminous macadam		1.23	1.24	1.20
Gravel	1.43	2.1	1.64	1.6
Earth	2.10	2.03	2.04	2.20

[1] "Rural Highway Pavements," McGraw-Hill Book Company, Inc.

TABLE 25B.—GAS CONSUMPTION ON DIFFERENT PAVEMENTS[1]

Type of pavement	Gallons per ton-mile	Miles per gallon, 2-ton truck
Earth (average conditions)	0.083	6.0
Gravel (average conditions)	0.059	8.5
Macadam (average conditions)	0.048	10.5
Rigid pavements (average conditions)	0.046	11.0

[1] "Rural Highway Pavements," McGraw-Hill Book Company, Inc.

TABLE 26A.—MOTOR TRUCKS. APPROXIMATE RELATION OF EACH ITEM OF COST TO THE COST OF GASOLINE[1]

Cost items	% of total	Ratio to fuel cost
Gasoline	12.9	1.00
Oil	1.9	0.12
Tires	9.2	0.71
Maintenance	12.7	0.99
Depreciation	14.3	1.11
License	1.3	0.10
Garage	3.7	0.29
Interest	3.8	0.30
Insurance	4.0	0.31
Supervision	6.7	0.52
Driver	29.5	2.29
Mileage items M	51.0	3.93
Time items T	49.0	3.81
Combined items	100.0	7.74

[1] *Bull.* 69, Iowa State College (1925).

TABLE 26B.—MOTOR BUSSES. APPROXIMATE RELATION OF EACH ITEM OF COST TO THE COST OF GASOLINE

Items	% of total	Ratio to fuel cost
Gasoline	14.8	1 00
Oil	1 8	0.12
Tires	11 2	0 75
Maintenance	15 6	1 06
Depreciation	13 9	0 93
Driver	26 4	1.78
All other items	16 3	1.09
Mileage items M	57.3	3 86
Time items T	42 7	2 87
Combined items	100.0	6.73

TABLE 26C.—APPROXIMATE RELATION OF EACH AUTOMOBILE COST ITEM TO THE COST OF GASOLINE (ORDINARY UTILITY AUTOMOBILE)

Items	% of total	Ratio to fuel cost
Gasoline	15 7	1 00
Oil	3.0	0 19
Tires	9 5	0 605
Maintenance	12 1	0.77
Depreciation	30 8	1.96
Interest	12 1	0.77
Insurance	3.0	0.19
Garage	8 1	0 52
License	5.7	0 36
Mileage items	71.1	4 53
Time items	28 9	1 84
Combined items	100.0	6 37

Summary of Motor Traffic Considerations.—From a practical standpoint, the following general conclusions seem sound:

1. The selection of maximum grade within the bounds of standard practice is not affected by the ability of single-unit motor vehicles to climb. The long trailer system demands low rates of ruling grade.

2. The selection of maximum grade up to 7% maximum is not affected by the factor of safe descent from the standpoint of single-unit motors, provided the alignment is nearly straight. 5% is desirable on curved alignment.

3. For a fixed rise and fall and distance a combination of different rates of grade has no appreciable effect on fuel consumption. The total cost of motor operation, including the time factor, however, is probably slightly less for a uniform grade. This effect is not, however, noticeable enough to reduce the steepest grade below a reasonable maximum and has no practical effect on the use of

TABLE 27.—ANALYSIS OF RELATIVE OPERATING COST OF AVERAGE MOTOR TRAFFIC FOR 1 MILE OF DISTANCE ON DIFFERENT RATES OF GRADE, BASED ON 11 CTS. PER MILE ON A 1% GRADE[1]

Distance factor, 2 cts. for 1% grade			Fuel factor, 3 cts. for 1% grade		Time factor, 6 cts. for 1% grade		Total operating cost, cents
Rate grade, per cent	Factor ratio for each rate of grade	Cost for each grade, cents	Factor ratio for each rate of grade	Cost for each grade, cents	Factor ratio	Cost for each grade, cents	
+10	1	2	4	12	4.0	24	38.0
+ 9	1	2	3⅔	11	3.6	21.6	34.6
+ 8	1	2	3⅓	10	3.2	19.2	31.2
+ 7	1	2	3.0	9	2.8	16.8	27.8
+ 6	1	2	2⅔	8	2.4	14.4	24.4
+ 5	1	2	2⅓	7	2.0	12.0	21.0
+ 4	1	2	2.0	6	1.6	9.6	17.6
+ 3	1	2	1⅔	5	1.3	7.8	14.8
+ 2	1	2	1⅓	4	1.1	6.6	12.6
+ 1	1	2	1.0	3	1.0	6	11.0
Level	1	2	0.9	2.7	1.0	6	10.7
− 1	1	2	0.8	2.4	1.0	6	10.4
− 2	1	2	0.6	1.8	1.0	6	9.8
− 3	1	2	0.4	1.2	1.0	6	9.2
− 4	1	2	0.2	0.6	1.0	6	8.6
− 5	1	2	0.1	0.3	1.0	6	8.3
− 6	1	2	0.1	0.3	1.2	7.2	9.5
− 7	1	2	0.1	0.3	1.4	8.4	10.7
− 8	1	2	0.1	0.3	1.6	9.6	11.9
− 9	1	2	0.1	0.3	1.8	10.8	13.1
−10	1	2	0.1	0.3	2.0	12.0	14.3

[1] "Location, Grading and Drainage of Highways," McGraw-Hill Book Company, Inc.

NOTES.—1. If anything, the fuel factor between the grades of −3 and −10% is a trifle low.

2. The time factor between the rates of grade of −5 and −10% depends very largely on the alignment, the individual driver, and whether or not he is in the habit of driving on steep grades. It is probably about right for passenger cars but too high for truck operation and might be reduced to 1.5 at a −10% for truck traffic in hilly country.

3. The time factor on grades of +2 to +10% is too high for high-power touring cars, as on good alignment the normal speed is often not reduced at all up to 6%.

rolling grades on intermediate profile design, as the value of smoothing out minor grade irregularities becomes less as the rate of grade is reduced.

4. For a fixed rise and variable distance depending on the rate of grade the lower the rate of grade the higher the fuel consumption and operating cost. Under these conditions the grade should be kept to the steepest reasonable rate.

5. In the matter of convenience in driving it is desirable to avoid shifting gears. The limiting rates of grade at which the gears are shifted for the ordinary car on improved roads is about 6 and 10% for pleasure cars, and 5 and 8% for standard trucks. This, however, is subject to constant change and is not of much importance.

6. The value of distance saved can be closely approximated.

7. The value of rise saved cannot be closely figured, but it is certain that it has more money value on steep grades than on light grades.

8. In locating roads, distance can be balanced against rise, but it is not possible to analyze this closely, and, as a rule, distance should rarely be increased, particularly if good alignment is lost, unless it is necessary to get a reasonable maximum grade or unless a noticeable localized rise and fall can be eliminated by a short additional distance. (Use Table 6, p. 12, for comparison of this nature.)

9. Ruling grades need not be consistent in rate so far as ordinary motor traffic is concerned as they do not limit the load of single-unit hauling rigs. Ruling grades should be consistent if the trailer train system controls the design.

It can be seen that the requirements of ordinary motor traffic have less definite claims for consideration in reducing the rate of maximum grades than horse traffic, but have more claim than horse traffic in the matter of reducing distance and rise and fall on account of the large amounts of money annually spent on operation costs. Summarizing, it can be said that ordinary motor traffic warrants higher rates of maximum grades than horse traffic, but demands short distance and less rise and fall on steep grades. The reduction of rise and fall on light grades is of very little practical value. For certain special conditions where the trailer train must be considered, maximum grades may well be reduced below even the limits required for horse traffic.

Practical Considerations Governing Selection of Grade

Effect of Maintenance Cost on the Selection of Maximum Grades.—The maintenance of shoulders, ditches, and water-bound macadam, gravel, or natural-soil surfaces increases in cost rapidly on grades over 5%. From the standpoint of maintenance cost 5% is the logical maximum rate.

Effect of Safe Footing on Maximum Grade.—In the matter of safe team footing, it is possible to select some type of pavement which will satisfy this condition for any grade used, but a change in surfacing to meet this requirement is often omitted on account of expense and more often by careless design. Most of the rigid-pavement types give satisfactory footing up to 5%, which is the practical limit without special design. Bituminous macadams can, by variations in manipulation, be made suitable for grades up to 8%. Plain macadams give good footing for any grade, but are too expensive to maintain over 5%. From the standpoint of team footing, 5% has a distinct advantage on main roads where rigid types are desirable, and 7 or 8% is a reasonable limit on side roads where some form of macadam or gravel will probably be used. Team footing is, however, becoming less important as a deciding factor. Seven per cent is a reasonable maximum from the standpoint of safety of motor traffic, due to skidding on a smooth pavement when brakes are applied. The following table indicates current practice in the matter of maximum grades for different surfaces.

SURFACE MATERIAL

	%
Wooden block..............	2
Asphalt block.............	4
Brick (grout joints)...........	5
Brick (mastic joints).........	8
Concrete[1].................	5 to 7
Bituminous macadam with flush or squeegee coat.....	5
(In sandy country, 6% when coarse sand is sprinkled on surface.)	
Bituminous macadam without squeegee	8
Water-bound macadam....	8
"Hillside" brick.	12
Stone block with open joints	12

[1] Hard to construct on grades over 5%.

Effect of Farm Hauling on Maximum Grade.—From the standpoint of accommodating ordinary farm team loads, 7% is the logical ruling rate. This is based on a load of 5000 lb. for farm hauling, which includes wagon weight. The records of produce dealers in the eastern states show that the ordinary wagon weighs about 1350 lb. and that 3500 lb. is a large net load. This load of 2.4 tons corresponds with the maximum theoretical load for 7% hard-surfaced grade. Team loads of 6 tons would be very unusual, which means that the ideal teaming grade of 2½% need not be considered except in level country where it can be obtained without much extra cost.

Effect of Construction Cost on Maximum Grade.—From the standpoint of construction cost 5 to 7% can generally be built without excessive expenditure even in hilly country.

Maximum Grades in Present Use.—The maximum grades in present use represent the best judgment of engineers from all over the world backed by practical experience and traffic tests of generations. It is true that they are largely based on factors of horse traffic, reasonable construction, and maintenance costs, but it is believed that these factors are still the most important deciding elements in the selection of maximum grade for most roads. Table 28 gives the rates in common use and is probably the most reliable basis for design that can be used.

TABLE 28.—MAXIMUM GRADES IN FOREIGN COUNTRIES

Location	Mountainous districts, %	Hilly districts, %	Level districts, %
Prussia....................	5	4	2½
Hanover..................	4	3⅓	2½
Baden....................	8	6	5
Brunswick..................	5½	4	3
Holyrood Road in England....	6	3½	

Military highway over the Alps, Italian side, 4½%; Swiss side, 6%.

Location	National roads, %	Departmental roads, %	Subordinate roads, %
France	3	4	6

MAXIMUM GRADES IN THE UNITED STATES

State	Main roads, %	Side roads, %	Unusual cases, %
New York	5 and 7	7 and 8	11
Massachusetts	5	7	
Connecticut	5		
New Jersey	5	6 and 7	9
Michigan	6		
Missouri	5 and 6		
Washington	5	5	
Illinois	6		9

UNITED STATES NATIONAL FOREST ROADS (MOUNTAINOUS DISTRICTS)

First-class roads . long grades 5%, short grades 7%
Second-class roads .long grades 7%, short grades 10%
Third-class roads. . . .long grades 10%, short grades 12%
State of Colorado (main mountain roads)....................6%

Effect of Maximum Grade on Cost.—Money spent on the reduction of maximum grade is never wasted unless distance is increased for a fixed rise by a grade lower than a reasonable maximum. It is not good policy to spend large sums to reduce below 5% in hilly country or 2% in level country, even where distance is not increased. The effect on cost of the selection of a 5 in place of a 6% or a 6 in place of a 7% depends largely on the method of construction that must be used. Where locations are fixed by well-established right of ways and permanent structures and the cost of new right of way is very high, grades are generally reduced by cut and fill. Under these conditions the effect of the selection of rate is very marked and no general relation can be established, as each case is a law unto itself.

Unfortunately, many of the roads in the older states were not laid out on natural engineering locations, and grade improvements are expensive either on account of excessive cut and fill or the high cost of new right of way on a better location. In mountain road or ordinary locations in newly settled districts, the question of right of way rarely handicaps the design and easy grades are obtained at moderate cost by natural locations which avoid steep adverse grades by going around a hill or develop moderate grades on a long climb by a longer distance. In climbing on a side-hill location the road section is generally what is known as a balanced section, that is, the cut just makes the fill by side displacement. The amount of excavation per mile is not affected by the rate of grade, but sometimes the length of road is affected.

Generalizing, it can be said that the effect of grade reduction on cost is not so marked as for cut and fill methods and that, roughly, the relation of cost to grade depends on the length, which is often inversely proportional to the rate; that is, where cut and fill is used a 5% grade might easily cost three or four times as much as a 6% grade, but where side-hill location is possible, a 5% would rarely cost more than six fifths as much as a 6%. This is, of course, affected

FIG. 11.—Balanced sidehill section.

by all sorts of local conditions and may not apply at all, but is true by and large and serves to illustrate the relation of rate to cost. To illustrate (Fig. 12): If the difference in elevation between *A* and *B* is 1000′ a 6% grade would require approximately 3⅓ miles of length and a 5% grade 4 miles to make the ascent. If the direct distance between *A* and *B* is less than 3⅓ miles the lengths of the two lines will be approximately as given. If the distance from *A* to *B* is more than 4 miles there would be little difference in the length, as it would merely mean that the 5% started to climb sooner than the 6%.

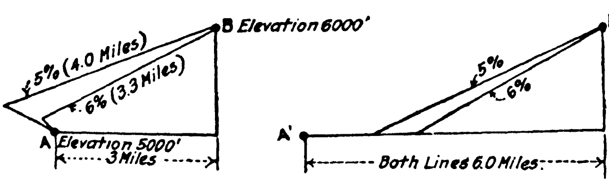


FIG. 12.

Under most conditions the cost would be more affected by the character of the excavation on the different locations and by the number of switchbacks required for the smaller rate. The difference in cost due to the difference in rate of maximum grade in mountain location does not often warrant the adoption of excessive grades.

Recommended General Practice Maximum Grades.—From the standpoint of horse traffic, single-unit motor traffic or trucks with one trailer, safe footing, and economy of construction and maintenance the following recommended rates of maximum grades will give moderately good satisfaction. In unusual cases the possibility of the extensive use of long trailer trains would tend to reduce these recommended rates, but the author wishes to emphasize the opinion that very few roads need be designed at this time primarily for long trailer trains. The following rates are satisfactory for the ordinary motor equipment used by the great majority of road users and additional expenditure would not be warranted for the benefit of a few men. For the effect of dangerous alignment on maximum grade see page 117.

Main Commercial Roads in Flat Country.—Long 2% ruling grades are desirable, but they do not justify much additional construction cost. Any long ruling grade up to 5% will probably be satisfactory. Short 6% are not inconsistent. A large volume of hauling by trailer trains might warrant reductions below usual practice, provided the interests operating such haulage paid the increased cost of construction.

Main Commercial Roads in Hilly Country (Well-settled Districts). Long 5% ruling grades are desirable and justify considerable expenditure, provided they do not increase the total distance. Seven per cent grades are probably justified to prevent increase in distance for a fixed rise. Long 6% grades are fairly satisfactory, but, as a rule, if 5% cannot be reasonably obtained it is just as well to jump to 7%. Short 7 or 8% grades are not inconsistent in connection with long 5 and 6% grades, provided the element of safe team footing is considered.

Main Roads Pioneer Districts.—Long 5% grades are very desirable, provided they do not increase the total distance, particularly if the road is a natural-soil road and considerable horse traffic prevails. Any long grade up to 7% is fairly satisfactory. Short 7 and 10% grades are not inconsistent except for trailer trains. Grades higher than 7% are not, however, in much favor on account of danger and high maintenance cost.

Side Agricultural Roads or Unimportant Pioneer Roads.—Any long grade up to 7% is satisfactory. Short 10% grades are consistent in connection with a 7% ruling, provided the element of safe footing is considered. Grades steeper than 7%, however, have a high maintenance cost and are dangerous even with good alignment.

Scenic Roads.—Long 6% grades are convenient on account of preventing gear shifts. Ten per cent is not unreasonable for such roads, except that on this grade the alignment should be easy, as later discussed, the maintenance cost is high, and considerable danger is added for grades over 7%.

Effect of Alignment.—Sharp curves affect steep grades, as taken up under the subject of Alignment (p. 117).

Unusually High Rates of Grade.—Grades as high as 11% have been constructed on state-improved roads in New York and as high as 9% in New Jersey and Illinois, but the general opinion of the departments under which these grades were built is that they would not again use such a high rate except in villages where any material change in street elevation would damage valuable properties. Outside of corporations it is bad practice to use long grades of greater rate than 7%, for if any road is of sufficient importance to warrant engineering plans for the future it is certainly of sufficient importance to warrant a reduction in grade to a reasonable rate.

Consistent Maximum Grades.—The design should be consistent if horse traffic is considered. Take, for example, a road between two shipping points. It is first necessary to determine the portion tributary to each terminal and then the practical grades on all the hills on each portion in order to decide what consistent ruling grade can be adopted without excessive cost. There is no object in reducing a hill from 7 to 5%, provided the total rise remains fixed with a large expenditure if near the terminal there is a grade that cannot be reduced below 7%. It should be borne in mind, however, that the nearer the shipping or market point is approached the more traffic the road will have, and if the hills are naturally flatter the ruling grade should be reduced. The direction of heavy traffic on each hill should be determined and considered. Ordinary motor traffic does not require consistent maximum grades, but the trailer train method does require them. Considerable expenditure is justified to obtain consistent grades for the benefit of team hauling on local service roads.

Intermediate Grades.—Intermediate grades include all rates between the maximum and minimum grades for the particular job in question. They afford the greatest chance for reasonable economy of earthwork of any part of the grading design and usually receive the least attention. From the standpoint of traffic they have practically no effect on travel cost or convenience on local service roads and only a slight effect on commercial roads. Economic analyses of relative value and relative grading costs show that it rarely pays to reduce intermediate grades below their natural rate (see Table 2, p. 6). Their proper use, however, controls the convenience and suitability of the road to abutting property and controlling conditions. In laying a profile grade the controlling points must first be considered; these are high-water levels of flood areas, elevations of existing bridges, railroad crossings, all points where deep cuts or high fills would damage the approaches to valuable property, connections with other highways, portions of the road previously improved, and, in villages, the elevation that will permit future widening and curbing that will fit the case.

Current practice handles most of these controlling features well with the exception of grades through villages, which are almost without exception too high for future widening and curb finish. Designers are cautioned to use city-street methods and to make the elevation the same as if a full-width curbed pavement was being designed.

Effect of Intermediate Grades on Cost.—All of these controlling points must be satisfied, but they usually affect only a small percentage of the length of any improvement and on the greater portion of the road the most economical elevation and any intermediate grade can be used. A grade so established that the cut in every cross-section would just make the fill at that point would result in the least possible excavation and the cheapest kind of grading methods. This condition can never be realized, but the nearer it is approximated the nearer the most economical grading design is approached. Where intermediate grades are applicable, there is no restriction on any combination of rates, as they have no effect on traffic-loads and very little effect on motor operation cost, and by an intelligent selection the ideal solution can be closely approximated. The cheapest and most satisfactory profile can be obtained by the use of the "rolling grade." By this is meant a profile made up of a combination of simple, compound, or reverse vertical curves, connected by tangent grades only when the tangent grade is the most economical or is necessary to prevent a series of short humps and hollows. Long, straight grades are not required, a mistake easily made by engineers trained in railroad work. Short grades are not objectionable and reverse vertical curves ride easily if well built. The rolling grade is also more pleasing in appearance than a straight profile if not carried to extremes. The detail methods of laying such a grade are described in Chap. XIV (p. 963). It appears

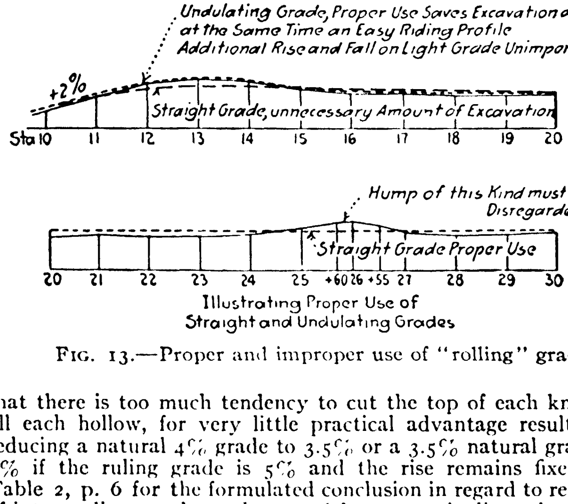


FIG. 13.—Proper and improper use of "rolling" grade.

that there is too much tendency to cut the top of each knoll and fill each hollow, for very little practical advantage results from reducing a natural 4% grade to 3.5% or a 3.5% natural grade to a 3% if the ruling grade is 5% and the rise remains fixed. See Table 2, p. 6 for the formulated conclusion in regard to reduction of intermediate grades and page 56 for economic discussion.

The importance of avoiding this tendency cannot be overestimated, as the plans of about 2000 miles of road, constructed in the last 10 years, which the writer has looked over in this connection show a needless expenditure of at least a million dollars for grading which has no practical value whatever. This element of costly design in current practice is probably due to the fact that the savings are not spectacular at any one place, but if the principle is consistently used the total result is spectacular.

It is also undoubtedly true that the previous railroad training of many road engineers has had a detrimental effect on intermediate profile design. From the standpoint of highway design, railroad practice overemphasizes the elimination of minor rise and fall on

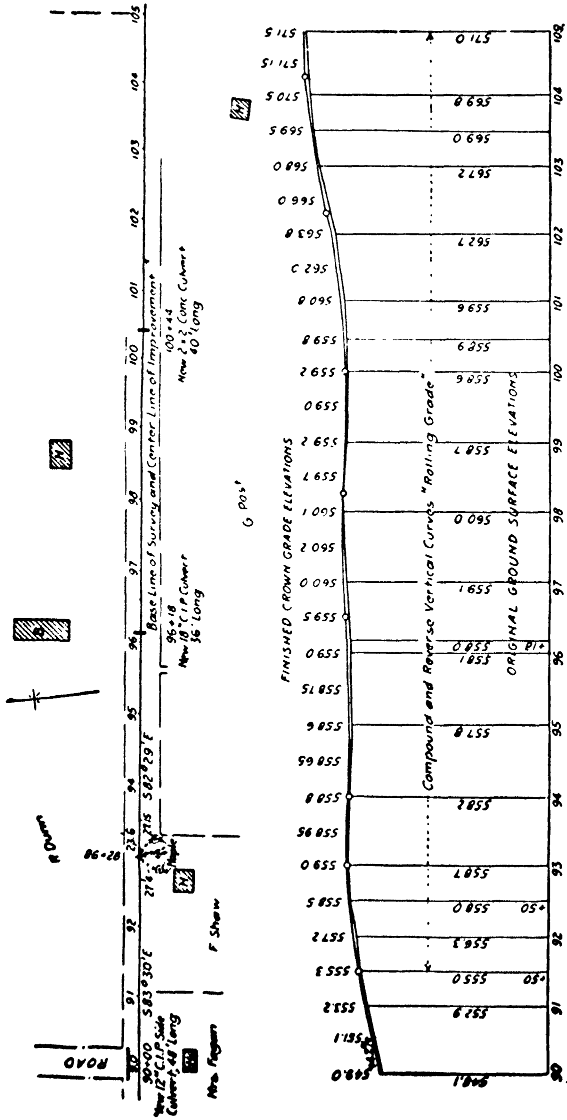


FIG. 14.—Typical example good use of "rolling" grade state road construction.

light grades. The author has personally applied the "rolling-grade" principle on construction work for the last 15 years and has found that the saving averaged about $500 per mile (using the 1926 scale of prices these savings would have averaged $1000 per mile). A systematic grade-line design will also often change the method of grading as well as reduce the yardage. To illustrate, the Heber Fruitland Road in Utah will be cited. The original design used long, straight railroad grades which required wagon haul; the redesign used a rolling grade which not only reduced the amount of excavation by about 30% but also practically eliminated wagon haul for most of the work and made it possible to handle the dirt with slip scrapers and road machine blade scrapers. This reduced the cost per cubic yard about 25%. The quantity reductions plus the unit-cost reductions amounted to approximately 50%.

In order to strengthen the force of the argument for "rolling grades" the following statement by G. R. Harr, Office Engineer of the Indiana Highway Commission, is inserted. The work to which he refers was done under the direction of H. K. Bishop, Chief Engineer.

"When we started here last May a year ago we had some plans previously prepared that had long, straight tangent railroad grades. We revised these plans using rolling grades having long and short vertical curves. In so doing we reduced the excavation very materially.

"From what I remember of the projects and the work on the same we saved from about 500 up to 4000 yd. to the mile. On one project the total excavation was cut practically in half."

The Effect of Arbitrary Profile Limitations on Cost.—A common grade-line limitation calls for tangent grades drawn to intersection with simple vertical curves easing off the apex and insists on 100′ of tangent grade between the ends of these vertical curves. This sounds scientific but has no practical value and is cited to illustrate the danger of ill-considered limitations. A specification of this kind often increases the grading by from 500 to 1000 cu. yd. per mile, an example of which is given below.

Pittsford North Henrietta Road in New York State
Length 2.67 miles

Original Design	Revised Design
Maximum grade, 5%.	Maximum grade, 5%.
Profile, straight grades with 100′ of tangent between vertical curves.	Profile, rolling grade.
Original amount excavation, 11,450 cu. yd.	Revised amount 9300 cu. yd.

(A saving of 800 yd. per mile.)

In conclusion, it may be said that the matter of intermediate grades needs more care than it often receives.

MINIMUM GRADES

Hard-surfaced Pavements.—Many road books claim that level grades should not be used because of the liability of water standing in ruts, and that a certain minimum grade should be adopted that will insure their longitudinal drainage. Baker states, in his "Roads and Pavements," that for macadam roads English engineers use a minimum grade of 1.5%, French engineers 0.8%, and that the American practice favors 0.5%. This means that:

For a 1.5% grade the fall would be 1/5" per foot.
For a 0.8% grade the fall would be 1/10" per foot.
For a 0.5% grade the fall would be 1/16" per foot.

The flattest crown that is ordinarily used even on bituminous macadam is 3/8" per foot, or twice as much as the greatest longitudinal fall in the above list. For long ruts the longitudinal grade is, of course, effective, but the patrol system of maintenance is supposed to prevent their formation and for short, small depressions the crown slope must furnish the drainage. There seems to be no reason why level grades should not be used on hard-surfaced roads; on such stretches the crown can be increased slightly to insure transverse drainage and the ditches given a minimum longitudinal fall of 0.2 to 0.5' per 100' depending on the soil to insure the longitudinal drainage of the surface water.

Earth Roads.—On earth or gravel roads attention should be given to minimum grades, as for these types they have some value, but not enough to warrant much expenditure.

It is advisable to use a 0.4 to 0.5% grade where much snow or rain occurs, but in the arid regions no minimum restriction should be specified.

ADVERSE GRADES

Adverse grades are defined as grades contrary to the general rise and fall of the road between terminals or controlling points. It is important to avoid them on mountain-road locations where the prime object is to gain elevation or on main commercial roads where the factor of rise and fall has considerable value. They are not a serious drawback for the usual road and cannot be avoided in ordinary rolling topography. This is so self-evident that it hardly seems necessary to state it. There is no serious objection to short adverse grades even on a long climb, if by their use the alignment can be bettered and excavation saved in crossing a small gully. There is no objection to adverse grades of 2% or less on any road. The main objection to long adverse grades is that they introduce considerable additional rise and fall which could be avoided by a better engineering location. This point is generally considered in the selection of the general route and is covered by the comparison of routes in the preliminary investigation.

VERTICAL CURVES

Vertical curves between tangent rates of grade add to the safety, convenience, and appearance of the highway. Vertical curves, as a rule, are picked out to fit the natural profile and in easy-rolling topography this method of selection need not, as a rule, be modified for any other consideration. There are cases, however, where the length of the vertical curve at the summit of a hill controls the length of sight ahead, and under these conditions certain minimum lengths are stipulated. A reasonable basis for decision in these cases appears to be founded on clear sight ahead at all times of 350′ for main commercial special-service roads and 250′ for local service roads except in mountainous regions where the sight distance requirement cannot be reasonably obtained. On this basis the following table is compiled, assuming that the line of sight is 5.5′ above the ground at the two ends and tangent to the vertical curve (see diagram for line of sight 5′ above ground at ends).

TABLE 29.—BASED ON A LINE OF SIGHT 5′ 6″ ABOVE THE GROUND AT BOTH ENDS

Algebraic difference in rates of grade, %	Minimum length of vertical curve in feet for a sight distance of 250′	Minimum length of vertical curve in feet for a sight distance of 350′
6	. .	.
8	.	150
10	50	250
12	135	330
14	190	400
16	225	450

As a matter of fact, merely on account of appearance and convenience in motor operation to prevent disagreeable checking in speed at the foot of hills, it is not wise to use a vertical curve less than 100′ long between grades having an algebraic difference in rate of 5% or less. Vertical curves are generally used between all grades having an algebraic difference of over one-half of 1%.

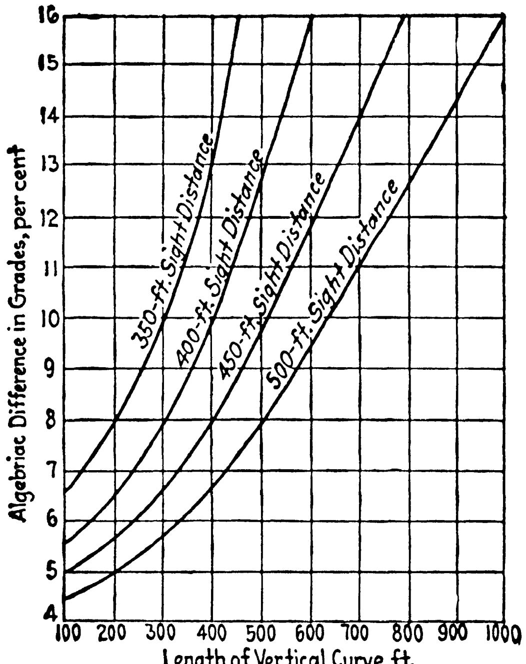
Minimum length of vertical curves from the standpoints of convenience, appearance, and sight distance can be assumed roughly as follows. There is no limitation on maximum length except that the curve should fit the profile without excessive grading. Vertical curves should be made as easy as possible without running up cost needlessly.

TABLE 30.—RECOMMENDED MINIMUM LENGTHS OF VERTICAL CURVES BETWEEN TANGENT GRADES

Algebraic difference in rates of tangent grades, %	Minimum length of vertical curves on local service roads, feet	Minimum length of vertical curves on special-service commercial roads, feet
5 or less	100	100–150
8	150	200
10	200	300
12	250	400
14	270	450
16	300	

Summary of Grades.—The discussion of economic grading design may be summarized as follows:

The road value of reasonable maximum grades and a minimum amount of rise and fall on steep grades cannot be overestimated. Any expenditure on these features is justified so long as it is consistent with the theory of cheap operation. The use of properly proportioned short maximum grades in connection with long ruling



Required length of vertical curves for different sight distances U. S. Bureau Public Roads Standard. (Based on line of sight 5 ft. above ground at both ends and tangent to road surface at middle.)

grades is a source of justifiable economy and works no hardship except for long trailer trains. The use of the highest reasonable maximum to shorten distance for a fixed rise results in considerable construction saving in many cases and is justified on the score of reducing motor operation costs. Distance should never be increased for a fixed rise to reduce grades below a reasonable maximum.

Minimum center-line grades have no road value on hard-surfaced roads and only a slight value on earth roads. Minimum ditch grades are important.

The traffic value of intermediate grades is negligible on local service roads and only of minor importance on special-service roads. Intermediate grade-line design has a large effect on grading cost and is entitled to very careful consideration. The most common faults of the ordinary treatment of these grades are the needless reduction of light natural grades and the use of long, straight railroad rates of grade. There is no practical advantage whatever from the use of long, uniform light rates of grade where the total rise and fall is not changed and very little real value is accomplished by the reduction of minor rise and fall occurring on light natural grades.

ALIGNMENT

Alignment affects the safety, speed, ease, and hauling power of traffic and the cost of road construction. Highway location is controlled at many places by the effect of curvature on maximum grade and the effect of alignment on construction cost. Sharp alignment modifies the allowable rate of grade, the width of pavement, width and shape of section, and increases the need for substantial safety devices, such as retaining wall or concrete or steel cable guard rail. At this point in the discussion it is necessary to consider only the factors in connection with alignment which would naturally control the field survey location, namely, the effect of alignment on grade, cost of construction, and safe sight distance for traffic. The effect of alignment on banking and widening pavements on curves and on the design of guard rail, etc. will be taken up in Chap. III.

In well-settled communities alignment is practically controlled by the existing road right of ways except where short relocations will materially reduce distance, needless rise, extreme grade, or danger to traffic. In sparsely settled communities alignment is not handicapped by right-of-way difficulties. As a general proposition, dangerous or crooked alignment should not be introduced to reduce grades below the maximum. If it is necessary in order to get the maximum grade or to keep the construction cost within reason, well and good. By this is meant that a straight road on a 5 or 7% grade is generally more satisfactory than the same road on a 3 or 5% grade with a dangerous turn. If the lower grade can be obtained without dangerous alignment and without increasing distance, all well and good. A reasonable maximum grade, however, should not be sacrificed on a side-hill location for better alignment, as future improvements, grading, etc. can reduce alignment danger much more easily than it can reduce grade by an entire relocation, and the danger at a few sharp bends on a long climb having normally safe alignment can be reduced by flattening the grade at the danger point; that is, the necessity of one or two sharp switchback turns to get a long reasonable maximum rate would not warrant an increase in grade above the reasonable maximum in order to eliminate these turns. Short maximums in connection with long lower maximums are warranted to improve alignment. The general principles of alignment are given on pp. 82 and 120.

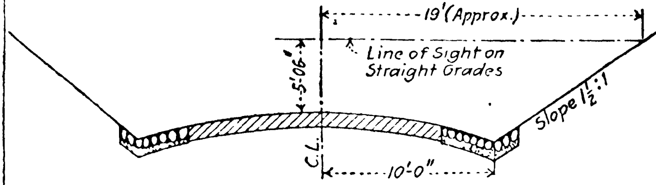
Dangerous Alignment.—Danger due to alignment is largely a relative matter. If a road is uniformily crooked, few accidents occur. The worst condition for accidents is a single sharp curve on a road otherwise straight. If an extremely sharp curve is necessary, the danger can be greatly reduced by gradually increasing curvature and reducing sight distance for other curves as the danger point is approached. This method has been successfully used in western New York in approaching difficult hair-pin turns.

The state of Maryland has kept track of auto smashups and reports more accidents on straight alignment than on curves and more accidents on sparsely traveled sections of main roads than in the congested districts near cities. This is, apparently, due to the temptation to "step on it" under apparently favorable conditions.

Sharp curves are not particularly dangerous for slow horse traffic but they are extremely dangerous for high-speed motor traffic, particularly on through roads used by drivers not familiar with the locality. It is, of course, impossible to protect traffic from the carelessness of speed maniacs, but the danger of collisions can be materially reduced by alignment which permits the driver to see ahead a reasonable distance at all times. Cars driven at high speed are liable to leave the road on almost any curve, as observation has shown that about as many cars go off moderately easy curves up to 500′ radius as they do on very sharp curves, on account of the tendency to take the easier curves at excessively high speed. The danger of collision is less, however, as it gives the other man a chance to protect himself. We are not particularly grieved if a fool does commit suicide. A touring speed of 25 to 35 m.p.h. is reasonable for main-road travel in ordinary rolling topography. Tests on the braking power of automobiles show that a passenger car traveling 20 m.p.h. can be stopped in 40′, and one going 40 m.p.h. in 140′ by the use of the emergency brake. As a matter of fact, brakes are not always efficient, a driver requires a little time to realize that danger exists after first seeing the approaching car, so that the determination of safe sight distance is largely a matter of judgment.

Sight Distance.—The authors have written to a large number of automobile clubs over the country and, in the main, they agree on 250 to 300′ as the minimum safe sight distance ahead at all times. The shorter distance is used on local service roads where most of the drivers are familiar with the road and the longer distance for main routes carrying foreign traffic not familiar with the existing alignment. This corresponds with the practice of various highway departments. Roads built with this limitation seem satisfactory to traffic. A sight distance of this kind does not necessarily depend on alignment unless the curve is in cut. Alignment is not affected by sight distance unless the curve is in cut or along a side hill where the cut slope is on the inside of the curve or where buildings or trees occur along the right-of-way line. The smallest radius of curvature that is permissible to give a certain sight distance depends on the width of the road section in cut and can be easily worked out diagrammatically for any special case. To give an idea of the various minimum alignment radii required for the different sight distances for curves in cut, Table 31, is inserted.

150' Sight Distance		200' Sight Distance		250' Sight Distance		300' Sight Distance		350' Sight Distance		400' Sight Distance	
A	B	A	B	A	B	A	B	A	B	A	B
Values given below are the radii in feet											
158	286	272	505	420	786	600	1130	815	1536	1062	2005



Contracted Section in Deep Cuts
This is the Section on which this Table is Based

TABLE NO 31

This table is compiled for the minimum width section used in New York State in 1919. Similar tables can be prepared for the standard sections in use in any particular locality. Column A in each case applies where the curve is on a straight grade and the line of sight is 5' 6" above the crown grade of the road. Column B in each case applies where the curve is at a change of grade and the line of sight is just above the ground at the ditch line.

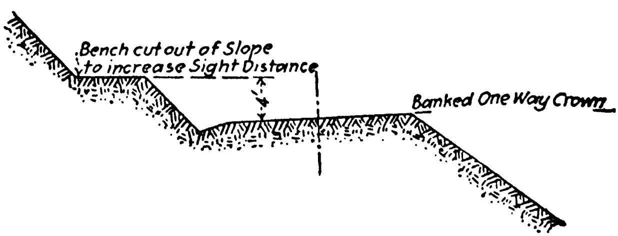


FIG. 15.—"Daylighting" a curve.

The radius for a specified sight distance can be figured by the formula

$$R = \frac{M}{2} + \frac{C^2}{8M},$$

where R = the road center line radius, in feet.

M = the distance in feet off the center line of the road where the line of sight comes tangent to the cut slope or any other obstruction.

C = required length of sight distance, in feet.

The sight distance for any specified alignment radius and standard section can be increased by "daylighting" the curve as shown in Fig. 15. This method has the distinct advantage of cheapening the grading cost and it also gives the driver a chance to see ahead even if he hugs the inside of the curve.

Current Practice, Minimum Curvature.—Sharp curves on steep grades or at the foot of such grades are not safe. Good practice calls for a minimum radius of 400′ to 800′ for these cases in ordinary topography. Right-angle turns even on level stretches are an abomination to the Lord. A minimum radius of 200′ for such cases increases the convenience of the road and is greatly appreciated by the road users.

France and Austria have used minimum radii of 100 to 165′ on main roads and as low as 50′ radii on district roads, but these limits are hardly suitable for fast traffic. The use of tractor trailer trains, four- or six-horse teams, or the hauling of long timber sometimes limits the radius of a curve.

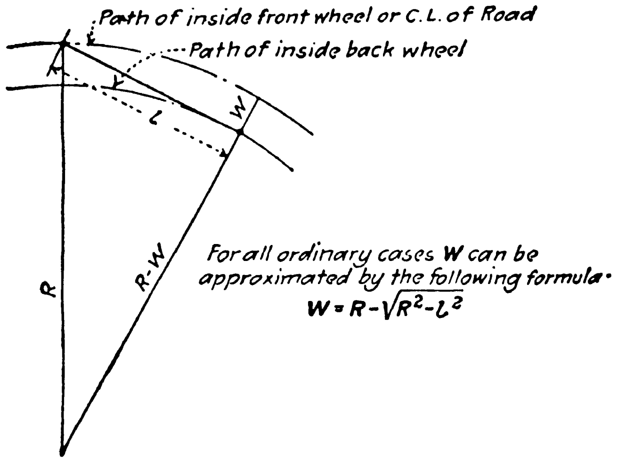


FIG. 16.—Rear wheel encroachment.

Rear-wheel Encroachment.—Under these conditions it is desirable to widen the section of the inside of the curve to provide clearance for the last wagon or the back wheels of a long rig as they work in towards the bank. To approximate roughly the distance that the last wheel works inside of the front guide wheel track, it will be assumed that the rig has a stiff connection between the front and rear axles. This will give a result on the safe side, as for a loosely coupled train or a special swinging rear axle much sharper corners can be turned.

TABLE 32.—TABLE OF APPROXIMATE ENCROACHMENT OF REAR WHEEL INSIDE OF PATH OF FRONT WHEEL FOR DIFFERENT LENGTHS OF RIG AND DIFFERENT RADII OF ROAD CENTER LINE, ASSUMING THAT THE CENTRAL ANGLE OF THE CURVE IS LARGE ENOUGH TO PRODUCE THE FULL ENCROACHMENT. THIS GENERALLY OCCURS WHEN THE CURVE IS TWO OR THREE TIMES AS LONG AS THE LENGTH OF THE RIG

Approximate radius of road center line, in feet	Length of rig between front and rear axle, in feet				
	10	20	30	40	50
	The values given below are the approximate distances in feet that the rear wheel runs inside of the front wheels				
40	1.3	5.4	...	....	....
50	1.0	4.2	10.0	....	....
60	0.8	3.4	8.0	15.3	....
70	0.7	2.9	6.8	12.6	21.0
80	0.7	2.5	5.8	10.7	17.5
100	0.5	2.0	4.6	8.4	13.4
120	0.4	1.7	3.8	6.9	10.9
150	0.3	1.3	3.0	5.4	8.6
200	0.2	1.0	2.3	4.0	6.4
300	0.2	0.7	1.5	2.7	4.2
400	0.1	0.5	1.1	2.0	3.1

NOTE.—According to Droune, the first pair of horses will occupy about 13′ ahead of the wagon and each additional pair 10′ more each. Wagons range in length from about 10′ for the bottom-dump type to 50′ for trucks hauling timbers. The ordinary commercial 5-ton truck has a wheel base of 14 to 17′. Recent regulations limit the length from 20 to 30′ overall, and the total length of trailer trains to 90′.

Mountain-road Alignment.—In mountain-road location it is generally impossible to provide a safe sight distance except for main routes as it would be prohibitive in cost on local roads. For such conditions considerable must be left to the care of the driver and the limitations of alignment are based more on the cost of construction than on the safety of traffic. Where long timbers are hauled over the road the foregoing table indicates the extra width or radius required.

Effect of Alignment on Grade.—On sharp curves it is desirable for the driver to have first-class control on the score of safety. An extremely sharp curve with a large central angle also reduces the hauling capacity of a six-horse team by from 20 to 40%. Considering both safety and team hauling, it is therefore desirable to reduce ruling grades on sharp curves. These considerations have no practical value on mountain roads for curves having radii greater than 100′, but on sharper curves good practice, recognizes this principle. Ordinary design uses radii of from 40 to 80′ on difficult switchback turns. For a 40′ radius the grade should not exceed 3% and for an 80′ radius 4% is a reasonable maximum. For high-class roads in well-settled districts at points where the sight distance is less than 350′ for commercial roads or 250′ for secondary state routes or local service roads, the grade should not exceed 3%. On high-class roads where the sight distance does not govern, the following simple rule has served satisfactorily.

Compensating Grades for Curvature (High-class Roads).—A simple rule for compensation of maximum grades on steep curves is to reduce grade around curve enough to permit future alignment straightening up to the highest standards of alignment without increasing the future rate of grade for the improved future location beyond a reasonable maximum. The authors compensate curves sharper than the permissible maximum for the class of road in question on the basis of first-class future alignment with reasonable maximum future gradient. This rule automatically takes care of the difference in central angle, which is an important factor.

Effect of Alignment on Motor Operation Costs.—Dangerous sharp alignment increases motor operation costs. It decreases normal speed and results in the needless use of second- or low-gear climbing and excessive braking on down grades. This action occurs,

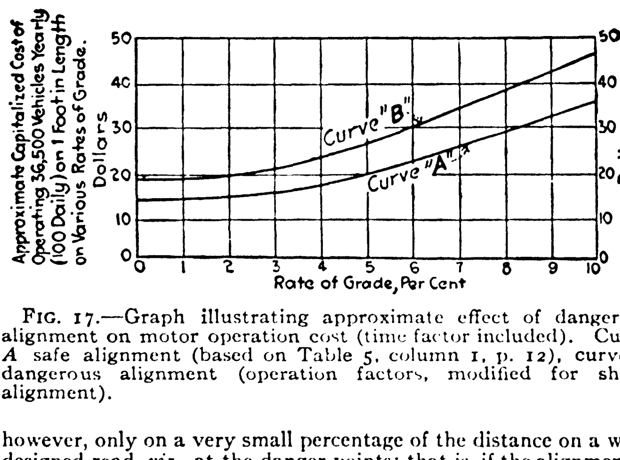


FIG. 17.—Graph illustrating approximate effect of dangerous alignment on motor operation cost (time factor included). Curve *A* safe alignment (based on Table 5, column 1, p. 12), curve *B* dangerous alignment (operation factors, modified for sharp alignment).

however, only on a very small percentage of the distance on a well-designed road, *viz.*, at the danger points; that is, if the alignment is safe for traffic it does not affect the operating cost. If it is dangerous for traffic it does affect the operating cost. As real danger is a more vital matter in the design than cost of operation, alignment design is controlled by the factor of safety and not by considerations of operating cost; that is, danger will be eliminated if it is possible to do so and no consideration of cheaper operation will improve the alignment if the consideration of danger had not been sufficient to warrant it.

While there are no good data on the effect of alignment on operating cost, there is sufficient general data to warrant stating with reasonable assurance that, if the radii of curvature are not sharper than from 250 to 300′ on the level or less than 400 to 600′ on grades and the sight distance is not less than 250′, reasonable speeds need not be reduced on account of alignment and motor operating costs are not materially affected. For sharper curvature and shorter sight distance the cost of operation is probably increased How much is not known, but purely as a matter of academic interest the factors used in compiling Table 5 (p. 12), for usual alignment, have been modified to conform with certain observed speeds on dangerous alignment. Figure 17 page 118 shows the result.

Curve *A* represents safe alignment, curve *B* represents dangerous alignment.

The general conclusion to be drawn is that motor operation is cheaper on good alignment and that, if alignment can be made safe by steepening the rate of grade, a slight increase in rate will not add to motor operation costs over that required for the lower rate and poor alignment.

In conclusion, it may be said from the standpoints both of safety and of operating costs that it is desirable to design special-service commercial roads for a sight distance of about 350′ and local service or secondary state routes for about 250′. Considerable expenditure is justified to obtain this requirement, but large additional cost in order to increase further the sight distance is wasteful and poor engineering, particularly on roads of secondary importance. This caution is not a needless one, as designers often excuse excessive profile and grading on the score that it increased the sight distance beyond the 350′ limit. This limit is not necessarily proper or liberal enough for all conditions, but it seems good sense to arrive at some limit suitable for the road in question and then to eliminate additional expenditure for an additional sight distance which may be fine if it can be afforded, but which is really not necessary. The tendency of almost all departments working with large appropriations is gradually to increase the fancy extras which may not amount to much for one case but which grow in number like a snowball until it is wondered why the cost of roads is going up and the mileage for appropriations coming down.

Effect of Alignment on Construction Cost.—For high-class road improvements in ordinary topography, alignment does not have much effect on cost of construction. There is no particular object in long tangents, and where an old road is being paved it is just as well to shift the center line slightly to keep on the old traveled way and take advantage of the old grading and any hard metalling that may have been placed in the past. Slight variations from the center of the right of way often save some grading expense and improve the character of the subgrade for the pavement.

In mountain-road location, alignment is given careful consideration, as it has a marked effect on cost. The radii are made as large as possible to fit the mountain side without excessive grading. On steep slopes the grade contour must be followed closely. There is no hesitation in using radii as sharp as 80′ at the head of gullies where the driver can see across the curve or a radius of 100′ on the outside curves around points where the sight distance depends on the radius. Even these limits are impracticable in very rough country where radii of 40′ are considered reasonable. All outside curves having a sight distance of less than 250′ should be posted with danger signs.

The arbitrary limitation of minimum radius has a large effect on cost. The following example will illustrate this point. These revisions were made by C. H. Chilvers on the Rabbit Ears Pass Road in Colorado to show the effect of alignment on excavation.

The office method of plotting a good cheap alignment is described in detail in Chap. XIV (p. 1108).

Rabbit Ears Road, State of Colorado, Side-hill Section

Original design	First revision	Second revision
Length, 8.79 miles Width of roadway, 16′ Maximum grade, 8 % Grades flattened on switchback turns Minimum radius, 100′ First-class alignment throughout	Length, 8.81 miles Width, 16′ Maximum grade 8 % No grade compensation on curves Minimum radius, 100′ First-class alignment but more curving, eliminating many expensive tangents	Length, 8.94 miles Width, 16′ Maximum grade, 8.5 % No compensation on curves Minimum radius, 40′ Poor, crooked alignment carried to extremes
Total amount of excavation, 91,000 cu. yd. First-class design but needlessly expensive	Amount of excavation, 65,000 cu. yd. First-class design shows effect of careful, intelligent alignment engineering	Amount of excavation, 38,000 cu. yd. Illustrates extreme effect of alignment on cost From an engineering point of view there was no justification for this design for the topography in question

Note.—On one switchback turn on this road a 100′ radius required 5000-cu. yd. excavation and a 40′ radius 500 cu. yd., or one-tenth as much. Short radii are justified in isolated cases but their continuous use to save small amounts is poor practice.

Effect of Railroad Grade Crossings on Alignment and Grade.—Railroad grade crossings are sources of continual danger; they should be eliminated on all main routes. Subway and overhead eliminations are discussed in Chap. IX. Specifications for approach grades and alignment are given in Chap. IX (p. 609).

Recommended Alignment Practice.—The following summary agrees with general current practice and can often be used without raising the cost beyond the bounds of reason. A summary of this nature is, of course, of only general value. Each case must be worked out on its own merits. Broad generalizations of detail requirements are dangerous if used indiscriminately.

Main commercial roads (well-settled districts):

Minimum sight distance........................ 300–400′.

Minimum radius of curvature at right-angle turns on level outside of villages where sight distance does not control.................................... 250–400′

Minimum radius of curvature on steep grades or at the foot of such grades, depending on the central angle where the sight distance is not the controlling factor. 600–800′.

Ordinary agricultural roads (local service):

Minimum sight distance.......................... 200–250′.

Minimum radius of curvature at right-angle turns on level outside of villages.......................... 100–200′.

Minimum radius of curvature on steep grades where sight distance does not govern.................. 400–600′.

Mountain roads:

No limitation on sight distance.

Warning signs used where necessary.

Minimum radius on steep grades..... ....100′.

Minimum radius in extremely rough ountry, 40′. Grades not to exceed 3% for a 40′ radius and 4% for an 80′ radius. Any grade up to 8% on a 100′ radius, although it is desirable not to exceed 5% on a 100′ radius curve with a large central angle.

Minimum Driveway or Intersection Radii:

Ordinary automobiles 25′ radius for turns of 90° or less

" " 35′ " " " over 90°

Trucks 50′ to 75′ center line radii

CHAPTER III

SECTIONS, PAVEMENT WIDTHS, RIGHT OF WAY, AND CLEARING

Introduction.—The shape and the width of road cross-sections affect the safety and convenience of highway traffic and they also affect the economy of grading design. It is desirable to obtain features that are fundamentally required for the satisfaction of traffic, but it is also desirable to avoid arbitrary standardization which adds materially to the cost without any adequate benefit. The problem of sections can be summed up as the determination of the minimum widths of grading, pavement, etc., the minimum depth of surface ditches in cut and variations in shape and width that will serve traffic requirements for the life of the pavement surface (10 to 15 years). Methods of estimating future traffic volume are given in Chap. I (p. 32).

At the time a road is improved, right of way should be acquired of such a width that it will permit the future widening of section, pavement, etc. Liberal right of way can be obtained more easily during the first stages of road improvements than at a later time when the land is worth more and buildings have been erected close to the road; *that is, right of way considers the future requirements of the road but the grading and pavement widths can only reasonably consider the requirements of traffic growth expected within* 10 *to* 15 *years.*

SECTIONS

Sections will be considered from the standpoints of safety, convenience, and economy.

Safety requires a grading shape that permits vehicles to use any part of the road from ditch to ditch without overturning or, if this is not possible, various expedients, such as the one-way crown, banking on curves, guard rail or wall protection, will very materially help the traffic. Safety requires a liberal sight distance, which on sharp curves can be obtained by "daylighting" the section (see Figs. 29, p. 155 and 15, p. 115).

Convenience requires sufficient width for vehicles to pass easily at any point in ordinary topography and provides special turnouts at short intervals on mountain roads. It also calls for crown and shoulder slopes that permit driving without an uncomfortable side tilt to the rig.

Economy of grading calls for various combinations of widths, ditch depths, back slopes, etc. which most nearly fit the natural conditions at all points; that is, the section must be flexible.

It is, perhaps, best to develop the discussion of sections and pavement widths, first, for high-class roads in ordinary topography, second, for pioneer roads in mountainous conditions; and, third, for village streets.

HIGH-TYPE ROADS

Premises of Design.—The points to be considered in the development of a normal section are:

1. What is a safe driving slope?
2. What is a comfortable driving slope?
3. What pitch is required to drain different surfaces?

These factors determine the shape of the section.

4. What is the commonly used width and the maximum width of the traveled way?
5. What is the minimum allowable depth of surface ditch?
6. What are stable slopes for cut and fill outside of the limits of the traveled section?

These factors affect economy.

The first three questions have been pretty well settled by current practice; the last three are not so well defined. The following premises will, however, be assumed, which can be modified for special conditions:

1. Three inches to one foot, or 4:1 is the maximum safe driving slope.
2. One inch to one foot, or 12:1 is the maximum agreeable driving slope.
3. Pavement crown should be kept to the minimum slope required for drainage of the pavement, as the flatter the crown the better distribution of traffic over the entire pavement area and the easier it is to drive a machine except on curves, where a banked slope is desirable to counteract centrifugal force. Normal crowns on straight alignment range from 1/8 to 3/4″ per foot, depending on the pavement surface. The practical limits of banking on curves range from 1/4 to 3/4″ per foot width.
4. The width of roadway subjected to hard wear on the lighter-traffic roads (single-track less than 300 vehicles daily) ranges from 8 to 10′ and on double-track roads (300 to 6000 vehicles daily) from 14 to 20′. The maximum width of roadway subjected to some wear by traffic turning out to pass ranges from 18 to 22′ and on heavily traveled roads repair parking requires at least 7′, shoulder width on each side of the pavement proper, or a minimum out-to-out shoulder width of 32 to 34′.
5. The minimum ditch depth below crown grade depends on keeping the longitudinal surface water outside of the traveled way and is rarely less than 10″; it depends largely on the amount of surface water that must be cared for.
6. The stable cut and fill slopes depend on the climate and the soil and range from 1/4:1 to 4:1.

Items 3 to 6 will be discussed briefly and typical sections derived.

Item 3. Pavement Crown.—The following discussion applies to rural highway pavements where the shoulder is flush with the pavement surface. For curbed city or village streets, see page 173.

Normal Crown.—Satisfactory normal crowns for different pavements on straight alignment have the following range:

Single-track, gravel, or water-bound macadam, ½ or ⅝″ per foot of half width. Circular arc recommended.

Double-track, gravel, or water-bound macadam, ⅜ or ½″ per foot of half width. Circular arc recommended.

Double-track, bituminous macadam, or oiled water-bound macadam, 5⁄16 to ⅜″ per foot of half width. Circular arc recommended.

Stone block or brick with bituminous joints on concrete base, 5⁄16″ per foot of half width. Circular arc recommended.

Sheet asphalt, brick with cement joints, asphalt block, and wood block, ¼″ per foot of half width. Circular arc recommended.

Cement concrete, ⅛″ per foot of half width (straight-line crown), or ¼″ per foot of half width (circular arc crown).

Banked Crown on Curves.—On sharp curves it is customary to use a one-way uniform banked crown to counteract the centrifugal force. A well-banked curve reduces the tendency to skid, increases comfort, and tends to keep traffic on the right side of the road, as no advantage accrues to the driver by cutting over onto the inside of the curve to take advantage of a steeper crown slope.

The proper amount of cross-slope considers the speed of traffic and the sharpness of the curve. It must be a compromise between the requirements of traffic traveling at the maximum legal speeds and slow-moving vehicles. It must also not exceed an amount which causes horses or slow-moving trucks to sideslip when the pavement is icy or which will cause heavily loaded trucks or hay ricks to overturn on account of their high center of gravity.

Centrifugal force is expressed by the formula $\frac{MV^2}{32.2\,R}$,

in which M = weight of vehicle in lbs.
V = velocity, in feet per second.
R = radius of curve, in feet.

See speed Table 113, (p. 616), for conversion of miles per hour to feet per second.

To get the theoretically correct cross-slope in inches of rise per foot width of pavement, the following formula can be used directly.

Bank slope rise in inches per foot of pavement width $= 0.37\,\frac{V^2}{R}$

$= 0.000065\ V^2D$,

where V = velocity of vehicle, in feet per second.
R = radius of curve, in feet.
D = degree of curve.

Table 33 shows the theoretical amount of rise per foot width for different speeds and curves to counteract centrifugal force entirely.

Table 34 shows actual cases satisfactory for normal pleasure traffic in summer and also cases where slow-moving vehicles and horse traffic have trouble during the winter months.

An examination of these two tables shows that slow-moving vehicles, particularly those with high centers of gravity, limit curve banking to ¾″ per foot of width and make it desirable to limit the slope to about ⅝″ per foot, particularly on steep hills.

From a practical standpoint it has been found desirable to bank curves on the basis of the theoretical requirement for a 25- to 30-m.p.h. speed up to the point where ¾″ per foot width is required, and for all curves sharper than this limit (600 to 900′ radius) use the same bank, ¾″ per foot, on the assumption that high speed should be and is reduced on such curves.

Table 35 gives a good practical basis for banking curves. This table must be modified by judgment where reverse curves occur or other special features, such as intersecting roads, etc. Figure 21 (p. 128) shows three methods of gradually warping the surface from normal crown to banked crown and of gradually widening the pavement on a curve to get a good practical transition curve. There is no object in road work in figuring a theoretical center-line transition curve similar to railroad practice.

TABLE 33.—THEORETICAL BANKING ON CURVES IN INCHES PER FOOT WIDTH OF PAVEMENT

$$0.37\frac{V^2}{R} = 0.000065\ V^2D$$

V = velocity in ft. per second.
R = radius of curve in feet.
D = degree of curve.

Radius of curvature, in feet	Banking for different speeds, inches per foot width			
	10 m.p.h.	20 m.p.h.	30 m.p.h.	40 m.p.h.
5730 = 1° curve	...	1/16	1/8	7/32
5000	...	...	5/32	9/32
4000	...	3/32	3/16	11/32
3000	...	1/8	1/4	7/16
2000	...	3/16	3/8	5/8
1500	1/16	1/4	1/2	14/16
1000	...	3/8	3/4	1 5/16
800	...	7/16	7/8	1 5/8
600	1/8	5/8	1 1/4	2 1/8
573 = 10° curve	1/8	5/8	1 1/4	2 1/4
400	5/32	7/8	1 13/16	3 3/8
300	1/4	1	2 3/8	4 1/4
200	3/8	1 3/4	3 5/8	6 3/8
100	3/4	3 1/2	7 1/4	12 3/4

NOTE.—For conversion of speed, miles per hour to feet per second, see Table 113 (p. 616).

TABLE 34.—EXAMPLES OF SATISFACTORY AND UNSATISFACTORY CURVE BANKING IN USE

Radius of curvature, in feet	Grade of road, per cent	Total bank	Bank slope, in inches per foot, width of pavement
Satisfactory banks for high-speed traffic with dry pavement surface			
1146 = 5°	Level	10″ in 16′	5/8
818 = 7°	Level	11″ in 21′	1/2
764 = 7° 30′	Level	11″ in 16′	3/4
674 = 8° 30′	Level	12″ in 19′	5/8
573 = 10°	0.8	15″ in 20′	3/4
286 = 20°	2.0	12″ in 16′	3/4
573 = 10°	5.0	9″ in 16′	5/8
498 = 11° 30′	6.2	12″ in 20′	5/8
Unsatisfactory banks for slow-speed trucks and horse traffic when pavement is slippery with light rain or sleet (sideslip occurs)			
573 = 10°	3.2	15″ in 20′	3/4
716 = 8°	7.0	12″ in 18′	2/3

TABLE 35.—PRACTICAL BASIS FOR BANKING CURVES, CONSIDERING BOTH HIGH-SPEED MOTOR TRAFFIC AND SLOW-SPEED TRAFFIC BOTH MOTOR AND HORSE DRAWN

Radius of curvature, in feet	Cross-slope rise, in inches per foot of pavement width		
	On grades less than 5%		On grades of 5% or steeper
2000	1/4	Theoretical bank 25 m.p.h.	3/16
1500	3/8		1/4
1000	1/2		3/8
800	5/8		1/2
600	3/4		5/8
400	3/4		5/8
300	3/4		5/8
200	3/4		5/8
100	3/4		5/8

NOTE.—The rates recommended must be modified by judgment where reverse curves or intersecting roads occur. Curves with a radius of more than 2000′ are rarely banked on main tourist roads. On local service roads carrying less than 1500 vehicles daily curves over 1000′ radius are rarely banked.

Variable Bank to Accommodate Both High- and Low-speed Traffic.—There is more and more demand for relatively high banking to accommodate high-speed motor traffic, and some designers hold to the 30-mile theoretical bank taking care of the slow-moving traffic by level stone-surfaced shoulders for double-track pavements and using a reverse parabolic cross-bank on pavements wide enough for four lanes of traffic, as shown in Figs. 19 and 20.

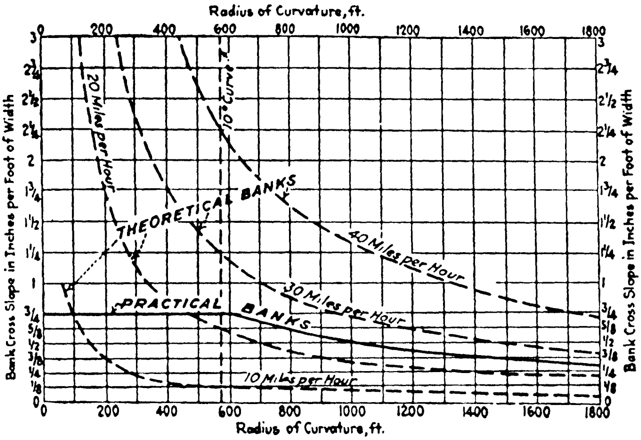


FIG. 18.—Graph of theoretical and practical bank slopes on curves.

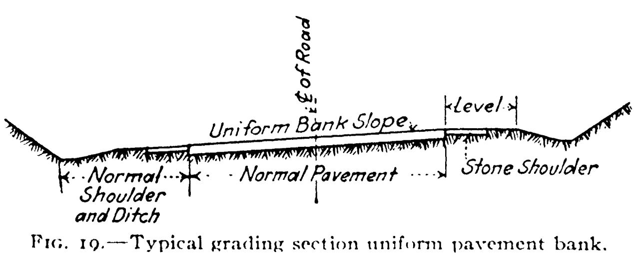


FIG. 19.—Typical grading section uniform pavement bank.

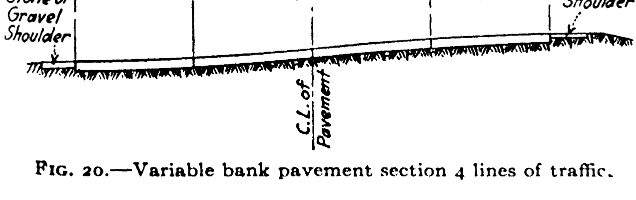


FIG. 20.—Variable bank pavement section 4 lines of traffic.

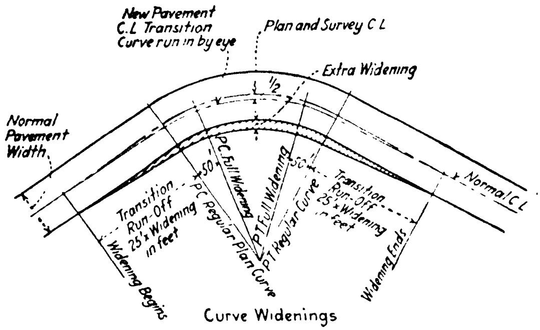


FIG. 21A.—Change from normal to banked crown at curves (Plan).

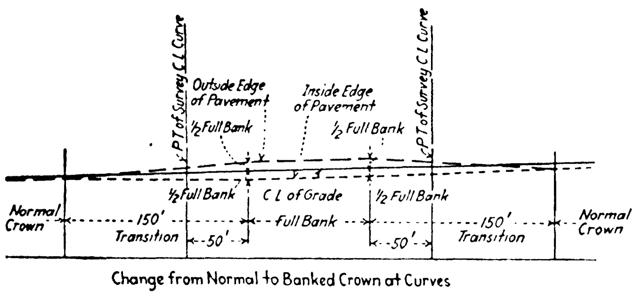
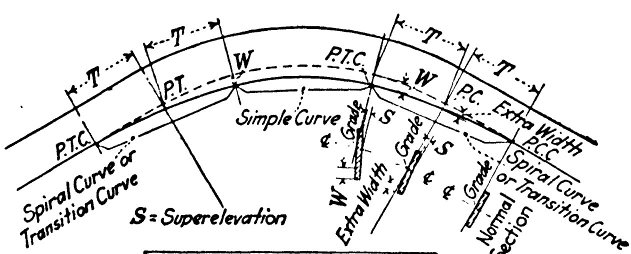


FIG. 21A.—Changing crown at banked curves (Profile).

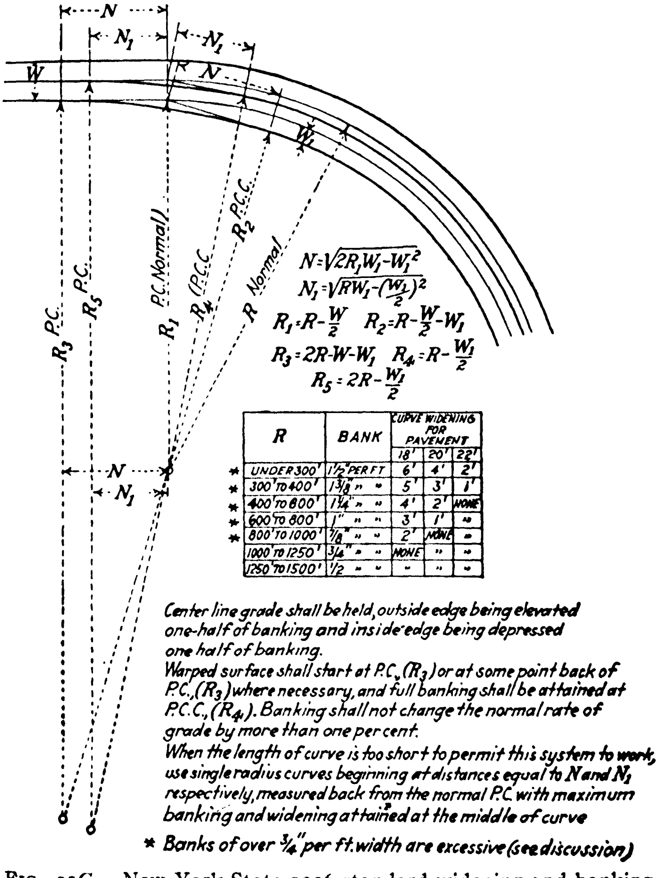


SUPERELEVATION AND WIDENING			
CURVE RAD.	SUPERELEVATION	EXTRA WIDTH "W"	DISTANCE "T"
UP TO 300'	1" PER FOOT	3'-6'	75'-100'
300'-500'	3/4" " "	1'-3'	50'-75'
500'-1000'	1/2" " "	0'	50'
CENTER LINE TO BE HELD AT THEORETICAL GRADE			

FIG. 21B.—U. S. Bureau of Public Roads 1925 standard widening and banking.

Changing Crown.—Figure 21 shows three typical methods of changing from normal to banked crown on curves.

The full bank coincides with the location of the full widening of the curve and extends from about 50′ beyond the theoretical P. C.



	R	BANK	CURVE WIDENING FOR PAVEMENT		
			18′	20′	22′
*	UNDER 300′	1½″ PER FT	6′	4′	2′
*	300′ TO 400′	1⅜″ " "	5′	3′	1′
*	400′ TO 600′	1¼″ " "	4′	2′	NONE
*	600′ TO 800′	1″ " "	3′	1′	"
*	800′ TO 1000′	⅞″ " "	2′	NONE	"
	1000′ TO 1250′	¾″ " "	NONE	"	"
	1250′ TO 1500′	½ " "	"	"	"

FIG. 21C.—New York State 1926 standard widening and banking.

of the regular curve to about 50′ short of the P. T. The transition from normal crown to full bank requires at least 150′ giving about two-thirds of the total bank at the P. C. and P. T. of the regular curve. The curve widening on the inside edge with center line shifted one-half of widening at all points gives a good practical transition curve.

The normal grade is carried around the center line of the pavement curve, the inside edge dropped, and the outside edge gradually raised as shown in Fig. 21. It is sometimes desirable to hold the inside at normal grade and bank entirely by raising the outside. Each case depends on local conditions and center-line grade profile, and the method should be left to the field engineer.

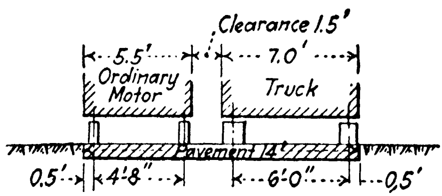
Current practice favors the bank slopes given in Table 35 (p. 126).

Item 4. Pavement and Shoulder Widths.—Pavement and shoulder widths are affected by volume of traffic, width of vehicles commonly using the road in question, type of pavement, sharp curvature, and the cost of construction and maintenance.

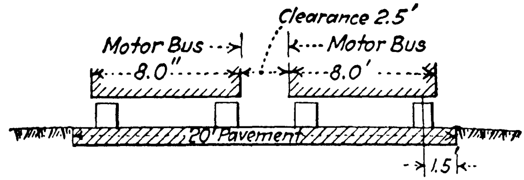
Volume of traffic classes the road roughly as single track, double track, or multiple track. Width of vehicles, safe clearances, and curvature control the width of traffic lanes on double-track or multiple-track roads. The ordinary passenger automobile has a body width about 5′ 6″ and a wheel gage of about 4′ 9″. Large trucks have a body width of about 7′ to 7′ 6″ and a wheel gage of about 5½ to 6′. Motor busses have a body width of about 8′. Traffic regulations generally limit the body width of vehicles to 96″ except traction engines, which may be 110″. Safe clearances are entirely a matter of volume of traffic and speed. For light-traffic roads where it is only necessary to turn out occasionally to pass and where such occasional passing justifies reduction in speed and careful driving it is proper to compute the pavement width on the basis of the outer wheel being placed within 6″ of the edge of the pavement or armored shoulder and allowing a clearance between vehicles of 1.5′ on straight alignment and 2 to 3′ on curves. For heavily traveled roads with continuous double-line or multiple-line traffic traveling at speeds of 25 to 40 m.p.h. the clearance between vehicles should not be less than 2½′ for ordinary cars or motor busses nor less than 2′ for trucks increased to 3′ on curves and the location of the outer wheel should not be closer to the edge of the pavement or armored shoulder than 2′ for ordinary autos or 1½′ for trucks.

Motor busses traveling at high rates of speed seem to be the controlling type of vehicle in establishing widths of traffic lanes on main highways (1926).

The following diagrams illustrate the width of pavements required on double-track roads for the two extremes of clearance outlined in the preceding paragraph.



Light-traffic Roads.—On these roads it does no harm if light vehicles turn off the pavement to pass, provided stone, gravel, or firm earth shoulders are provided for such turn-out traffic.



Main intercity roads, motor-bus requirements (straight alignment).

NOTE.—Diagrams on page 135 show distribution of traffic on pavements of different widths.

These conditions have resulted in the following common practice, which is summarized in Table 36 for straight alignment. Table 37 shows widths required on curves to allow for extra clearance and rear-wheel encroachment discussed on page 116. Single-track pavements (less than 300 vehicles daily) range in width from 8 to 12', depending on the type of pavement, 8' to 10' being used for the harder surfaces and 10' to 12' for gravel or sand clay. Double-track roads (300 to 6000 vehicles daily) range in pavement width

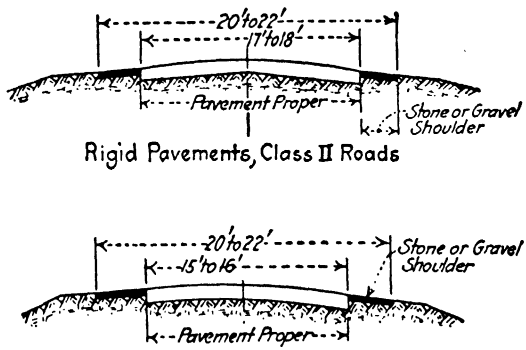


FIG. 22.—Comparative pavement and armored shoulder widths Class II traffic.

from 15 to 20' on straight alignment with special stone shoulders (see Fig. 22).

For rigid-type pavements on double-track roads a width of less than 18' is inadvisable on account of the formation of dangerous ruts along the pavement edge, the added cost of shoulder maintenance (see p. 525), and the undesirability of heavily loaded wheels traveling close to the outside edge, which increases the probability of corner crack failure (see design of rigid pavements, p. 368). For macadam pavements with stone shoulders it is permissible to cut below this width for traffic of less than 1500 vehicles daily.

For multiple-line traffic an allowance of 9′ per traffic lane is common practice for straight alignment widened on curves for rear-wheel encroachment (see p. 116). Ten feet per traffic lane is advocated by some engineers where there is much motor-bus traffic.

TABLE 36.—COMPARATIVE WIDTHS
(In feet)

Class of road	Vehicles per day	Rigid pavements		Macadam or gravel	
		Pavement proper	Out-to-out width armored shoulders	Pavement proper	Out-to-out width armored shoulders
Class I[a]	2000–6000	18–20	22–24	18–20	22–24
Class II	800–2000	17–18	20–22	15–16	20–22
Class III	300– 800	16	16	12–16	16
Class IV	Less than 300	.. ...	.. ..	8–12	

[a] For traffic of over 6000 daily (10-hr. count in summer) increase pavement width to 27 or 36′. See page 28 for discussion of road capacity.

TABLE 37A.—CLASS I TRAFFIC (SHARP CURVES)

Radius of road center line, in feet	Total pavement width, in feet	Length tangent run-off, in feet Fig. 21A, p. 128
100	25	100
150	24	90
200	23	90
300	22	80
400	22	80
500	21	70
600	21	70
800–1000	20	60

NOTE.—Normal pavement widths of 18 to 20′ used on all curves having a radius greater than 1000′.

TABLE 37B.—CLASS II AND III TRAFFIC (SHARP CURVES)

Radius of road center line, in feet	Total pavement width on curves for a double-track road (local service), in feet	Length tangent run-off, in feet Fig. 21A, p. 128
50	29	100
75	25	100
100	23	100
150	22	90
200	21	90
300	20	80
400	20	80
500	19	70
600–800	18	50

NOTE.—Normal pavement width (15 to 18′) used on all curves having a radius greater than 800′.

Changes in Pavement Width.—Special extra width is used on curves, at railroad crossings, at long span bridges, and through villages. In changing from narrow to wider widths the following rules have been found to give satisfactory results.

For curves widened on one side only where the center line of the pavement shifts one-half the total widening the length of transition from normal pavement width to extra-wide width should be at least 20 to 30′ for each foot of flare for the edge of the pavement (see Fig. 21, p. 128).

For widenings on straight alignment or curves approaching railroad crossings, bridges, etc. where the vehicle must swerve first to the right and then straighten out again, a distance of 30 to 40′, per foot flare of edge of pavement is required (see Fig. 193, p. 607).

Effect of Pavement Width on Cost.—Table 38 shows approximate construction cost per foot width of pavement per mile. It can be readily seen that where funds are limited it is desirable to select a reasonable minimum which will meet traffic requirements.

TABLE 38.—PAVEMENT COSTS

Type of pavement	Assumed cost per square yard	Cost per foot width 1 mile long
Brick	$4.50	$2700
Asphalt concrete	3.50	2100
Cement concrete	3.20	1920
Penetration bituminous macadam	2.20	1320
Water-bound macadam	1.80	1080

It is well to bear in mind that the pavement can always be widened by the maintenance or by reconstruction at a later period when the traffic volume increases sufficiently to warrant added width and that it is rarely advisable to select the width for a volume of traffic greater than expected in 15 years, as this is about the usual life of rural highway pavement surfaces.

Shoulder Width.—Shoulder width to permit turnout traffic depends on the same factors as pavement width with the added provision for standing rigs outside of the pavement area on heavily traveled roads.

Table 39 shows the results of Massachusetts investigations 1896 to 1900. These results were obtained under the old horse-drawn traffic conditions and do not apply closely for the conditions of today. They are included in connection with this discussion to illustrate the change which modern automobile traffic has made in width requirements on the heavier-traffic roads. They, however, show a general relation between areas of light and heavy traffic on the lighter-traveled agricultural roads.

The second part of Table 39 gives the results on a few roads, showing the form used and the variations from year to year. The footnote gives a summary of 160 roads and shows the results much better than by printing the table in full.

Stated briefly, the widths subjected to continuous wear on unimportant roads ranged from 8 to 10′, on well-traveled roads 10 to 14′, and in unusual cases, 14′ to 16′. The maximum widths for turnout traffic varied from 12 to 14′ on side roads and 17 to 18′ on the main roads.

Modern traffic has changed conditions on the main roads, but does not greatly affect these figures on the lighter-travel roads up to about 300 vehicles per day. Similar widths on New York State main roads have been measured and found to check the widths of heavy travel of 14 to 16′ but the maximum turnout widths were more, running from 20 to 22′. This can be explained by the increase in automobile traffic which, on account of its higher speed, requires more room in passing.

TABLE 39.—SHOWING WIDTHS OF TRAVELED WAY

Town or City	County	Width of Macadam	Maximum Width of Traveled Way				Width of Commonly Traveled Way			
			1896	1897	1898	1899	1896	1897	1898	1899
Athol	Worcester .	17′	16′	16′	20′	18′	10′–12′	12′	14′	14′
Barre	Worcester .	15′	—	13′	14′	14′	—	9′	7′	8′
Bedford	Middlesex .	15′	—	12′	15′	15′	—	8′	10′	9′
Chicopee	Hampden .	20′	—	20′	20′	20′	—	12′	12′	13′
Dalton	Berkshire .	15′	20′	20′	21′	16′–21′	20′	16′	18′	12′–18′
Fitchburg (W.)	Worcester .	15′	15′	14′	18′	18′	10′	10′	15′	14′
Huntington ..	Hampshire.	15′	9′	11′	11′	12′	7′	8′	9′	8′
Lincoln	Middlesex .	15′	15′	15′	15′	15′	10′	9′	10′	10′
Marshfield ...	Plymouth..	15′	14′	12′	11′	12′	8′	9′	7′	7′
North Adams	Berkshire .	15′	10′–12′	13′	14′	15′–20′	8′–10′	9′	10′	12′
Orange	Franklin ..	17′	16′	16′	20′	20′	10′–12′	12′	15′	15′
Taunton	Bristol	15′	20′	20′	15′	18′	10′–15′	10′	8′	7′–12′

Width of traveled way on 160 roads in Massachusetts, measured during the years 1896, 1897, 1898, and 1899, and printed in the report of the Massachusetts Highway Commission for 1900.

The width of stone on these roads is given as 15′ wide on 130, 12′ wide on 3, and 10′ wide on 2. It should be remembered that the stone is put on very much thicker in the middle than at the edges.

The maximum width of traveled way as measured was as follows:

9 ft. wide on	2 roads	18 ft. wide on	23 roads
10 " " "	6 "	19 " " "	1 "
11 " " "	2 "	20 " " "	10 "
12 " " "	28 "	21 " " "	10 "
13 " " "	8 "	22 " " "	1 "
14 " " "	23 "	24 " " "	2 "
15 " " "	30 "	25 " " "	4 "
16 " " "	8 "	26 " " "	1 "
17 " " "	1 "	33 " " "	1 "

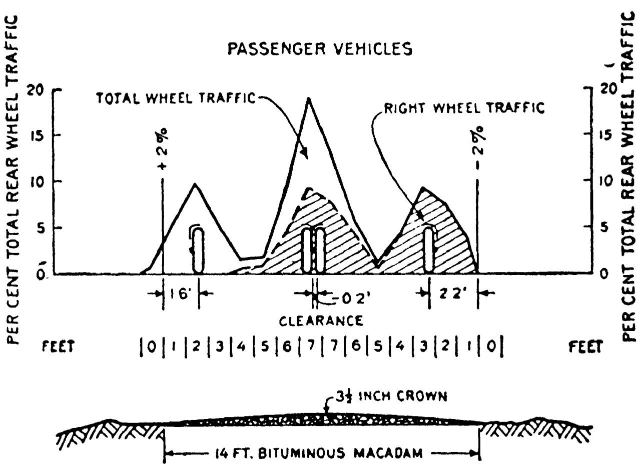
The width of commonly traveled way as measured was as follows:

7 ft. wide on	12 roads	14 ft. wide on	8 roads
8 " " "	17 "	15 " " "	13 "
9 " " "	25 "	16 " " "	2 "
10 " " "	32 "	18 " " "	4 "
11 " " "	10 "	20 " " "	2 "
12 " " "	30 "	22 " " "	1 "
13 " " "	3 "	25 " " "	1 "

Even a single-track pavement should have ample shoulder width to permit traffic to turn out and pass easily; that is, the total width of pavement and driving shoulder, no part of which should have a slope of more than 1″ to 1′, is practically the same for single- or double-track roads.

For roads having a volume of traffic of over 1500 vehicles daily a driving shoulder width of at least 7′ is required to permit parking, as a great many accidents have occurred due to vehicles standing on the pavement area of double-track roads.

The following diagrams illustrate typical transverse distribution of traffic on different width pavements. These diagrams are taken from an article by J. T. Pauls of the U. S. Bureau of Public Roads published in *Public Roads*, March 1925, and verify the foregoing text. This investigation of the U. S. Bureau is the most complete study that has been made in transverse distribution of traffic under modern traffic conditions.

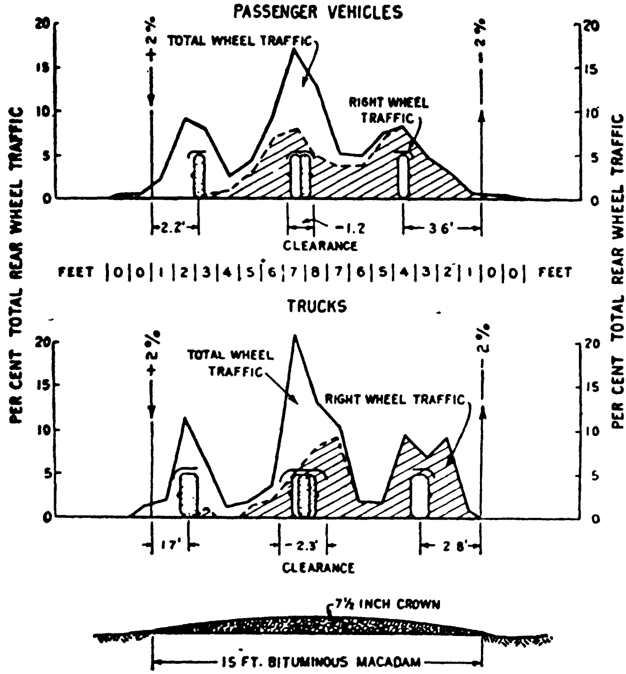


Test 9 shows the distribution of traffic on a 14′ bituminous macadam road. Note that the traffic spreads out into the shoulder at the left. This fact and the lower crown make the negative clearance on this road less than that of the 15′ road shown in Fig. 4.

Recommended Practice, Pavement and Shoulder Widths.—The available data obtained from observations on actual traffic movement indicate that a minimum turnout width of 20′ is desirable on single-track side roads, 22′ on secondary double-track roads, and 24 to 32′ on main double-track special-service roads; for a triple line of traffic 41′, and a four-track road 50′.

From these data it appears that modern practice on single- and double-track roads requires a width of solid pavement of from 10 to 20′ on straight alignment and a total driving width, including shoulders, of from 20 to 32′. These widths are modified for sharp alignment, as previously discussed.

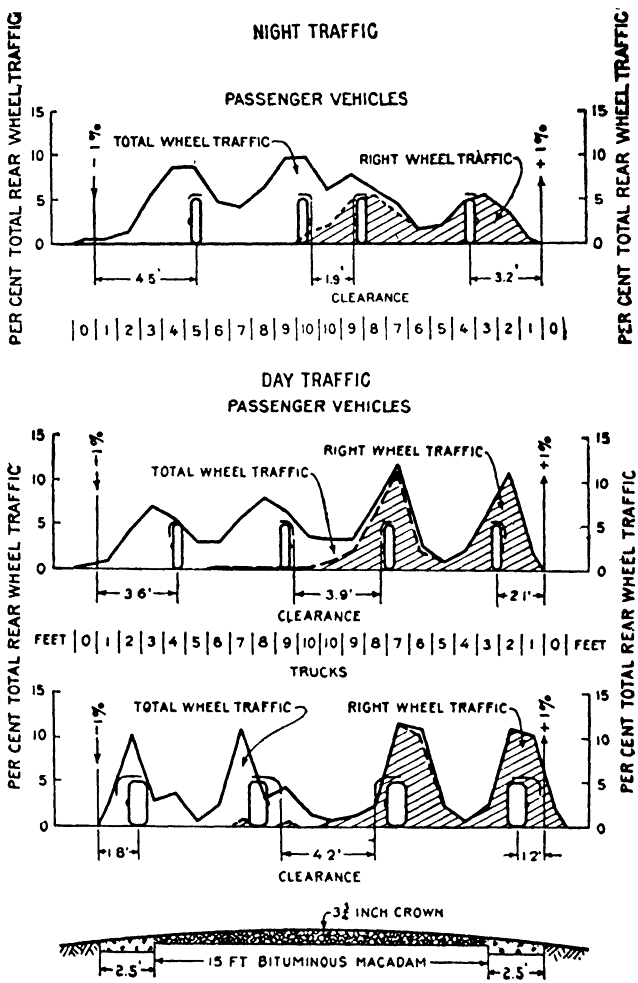
A standard for the portion of the section used for driving (Fig. 23) has now been practically developed. The pavement that is to carry the heavy traffic has a specified crown for each variety and ranges from ⅛ to ¾″ to 1′. The shoulder slope from the edge of



Test 7, made on a 15′ road, shows how the maximum concentration of traffic occurs at the center on roads of such narrow width. The steep crown in this case accentuates the concentration at the center; and the effect of a slight downhill grade is noticeable in the greater distance of the average wheel position from the edge on the downhill side.

the pavement to the limits of the driving width (20 to 32′) has a slope of 1″ to 1′ or possibly ¾″ to 1′; that is, the shape of the driving portion of the normal section is fixed. The flexibility of the section depends on the portion outside of this driving width.

The function of the extra width is to keep the longitudinal drainage of surface water beyond the portion used for driving. To do



Test 5 shows the separating effect of concrete shoulders on tangents and difference in the distribution of traffic at night. Compare with the concrete road of the same width shown in Fig. 23 which has a center line for the separation of the traffic.

this, there is a limitation of a minimum slope of 1″ to 1′ to insure transverse drainage and a maximum of 3″ to 1′ on the score of safety. It is by the good judgment of the designer in using various slopes between these limits and various widths and depths of ditches combined with the possibilities of different grades, that the economies in earthwork are effected and at the same time the design is made appropriate to the local conditions.

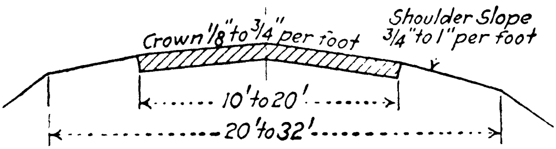
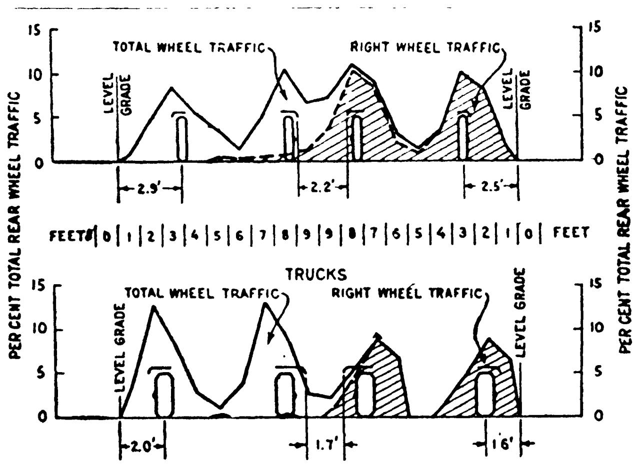
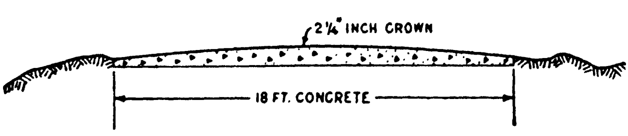
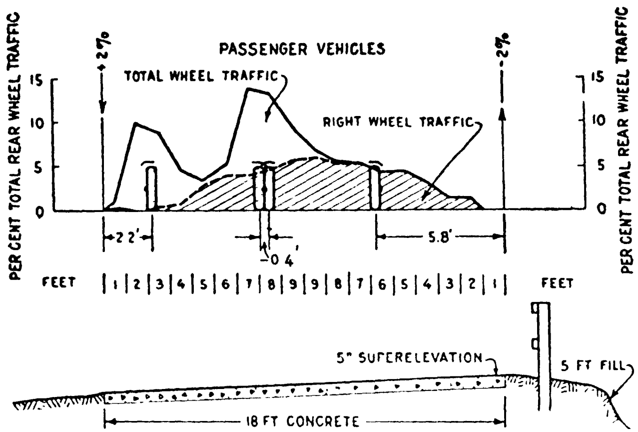


FIG. 23.—Normal pavement and shoulder section. (Traffic less than 5000 daily 10 hour count in summer.)

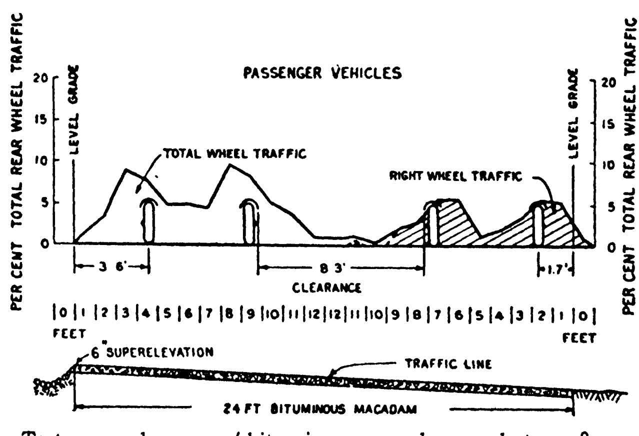




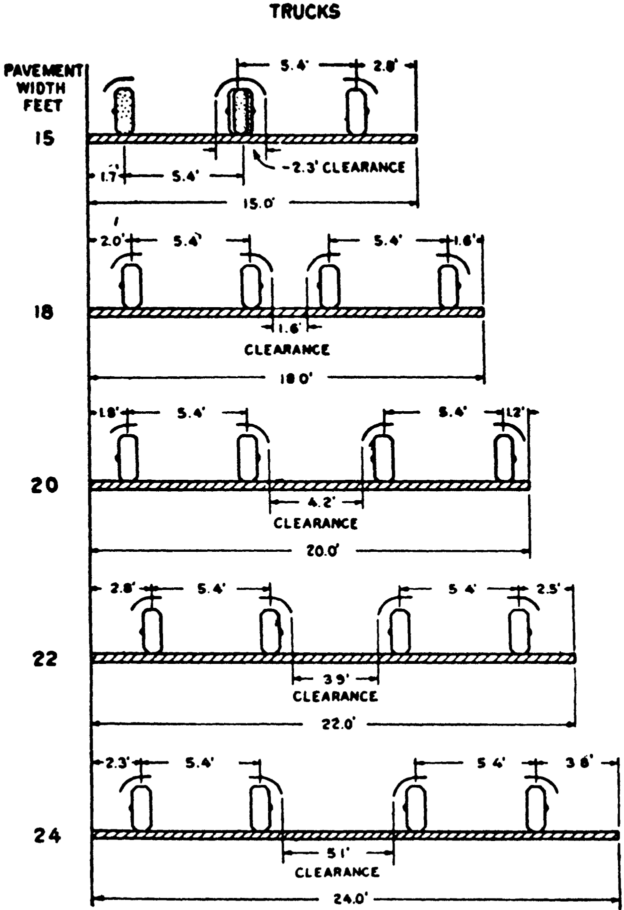
Test 4, on an 18′ concrete pavement, shows how the center traffic concentration which characterizes the narrow roads is relieved by widening. The good shoulders on this road encourage traffic to use the surface to the very edge. Contrast this condition with that shown in Figs. 15 and 16, where a bad shoulder condition clearly discourages the use of the edges of the surface.



Test 22, made on an 18′ concrete pavement at a 25° curve with plane superelevated section and no center line, shows the effect of the absence of the center line and the presence of a menacing fill at the outside of the curve on the distribution of traffic. The shift toward the inside of the curve is accentuated by the slight downhill grade at the outside.

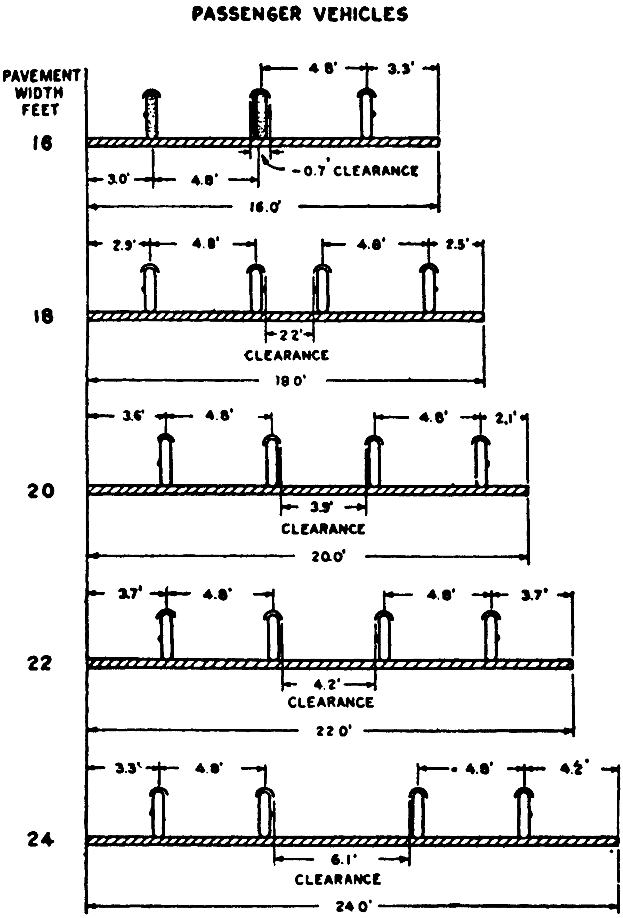


Test 19, made on a 24′ bituminous macadam road at a 20° curve with a plane superelevated section and center line, shows a better separation of the traffic than Test 16 (Fig. 9) made on a similar road with crowned superelevated section.



Clearance diagram for truck. Preferential driving location.

Average distribution of traffic on road surfaces of various widths from 15 to 24'. Eighteen feet is the minimum width which gives positive clearance for all vehicles in passing at the preferred position. The clearance increases with the width of the pavement, but becomes unnecessarily large when the width exceeds 20'. Widths of 22 and 24' are apparently excessive for two lines of traffic, and not great enough for three lines.



Clearance diagram for passenger autos for preferential driving position.

Average distribution of traffic on road surfaces of various widths from 15 to 24'. Eighteen feet is the minimum width which gives positive clearance for all vehicles in passing at the preferred position. The clearance increases with the width of the pavement, but becomes unnecessarily large when the width exceeds 20'. Widths of 22 and 24' are apparently excessive for two lines of traffic, and not great enough for three lines.

Item 5. Depth of Ditches.—The authors' experience indicates that an open ditch does not have much effect on ground water; that its part in the design is to drain the surface water, and that if ground water is encountered underdrains must be used. These conclusions have been borne out in practice and are advocated by many engineers, notably Irving W. Patterson of Rhode Island, who has had unusual success with his drainage and foundation designs. The principle to be emphasized is that deep surface ditches below the elevation of the bottom of the pavement foundations are useless. Deep ditches are not only useless but dangerous, and the best practice calls for the least depth of ditch that will handle the surface water.

A great many road men seem to feel that a deep open ditch really helps to drain the subgrade but, as stated, the author has never been able to prove by cases where foundation failure occurred that the depth of surface ditch had any well-defined bearing on the matter, provided the ditch carried away the surface water promptly. Some soils have a strong capillary action and the water works up

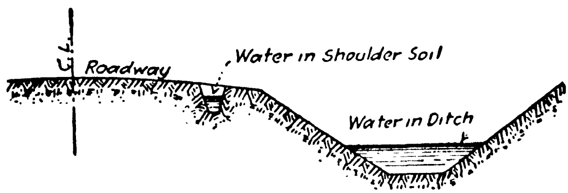


FIG. 24.

through them. In impervious soils, such as clay, a surface ditch 15′ from the center line cannot have much drawing action, as in numerous cases small holes dug in the roadbed (Fig. 24) fill with water at a much higher elevation than the side ditch.

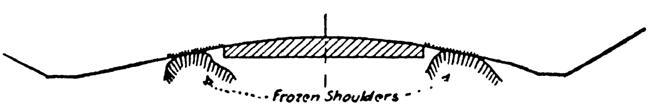


FIG. 25.

In many instances in the northern states the ground under the pavement proper thaws out before the shoulder material, which is protected by a sod coating, and the following result is obtained (Fig. 25). Under these conditions the moisture in the center is held even in porous soils. As a matter of fact, all pavement foundation design must be predicated on the assumption that, even with the best drainage schemes, the subgrade will at times soften somewhat, and for this reason the use of deep ditches which are inconvenient to traffic and which increase the grading cost are not in so much favor as in the past.

Frequent culverts are desirable to rid the ditches of excess water. It should be remembered that road ditches are to protect the road and not furnish farm drainage and that deep farm ditches should be kept away from the road section.

The following Rhode Island standard grading sections (Fig. 26) show the use of the shallow 12″ ditch which is advocated wherever a small amount of surface water is expected.

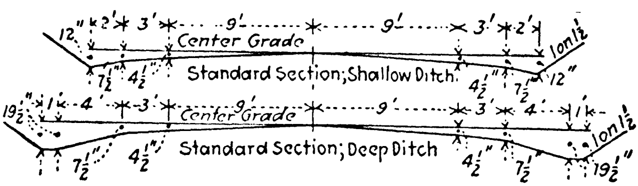


FIG. 26.—Rhode Island standard grading sections.

The following section (Fig. 27) represents a good typical minimum width and a minimum ditch depth grading section for single- or double-track roads which have been proved by practice to be satisfactory where small amounts of surface water are encountered. This section results in about the least feasible amount of cut and fill in grading design for light cuts and fills. The approximate carrying capacity of ordinary road ditches and the limitations of use of the

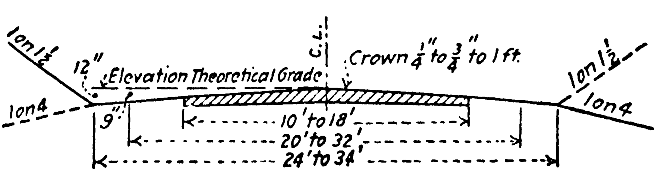


FIG. 27.—Typical minimum depth ditch section. (Traffic less than 5000 daily.)

shallow and medium road ditches are discussed under Longitudinal Drainage (p. 334).

Effect of Grading Width on Cost.—The width of grading from ditch to ditch has a distinct effect on cost, but no general relation can be established for the ordinary road improvement where an old road forms the basis for the new grading. Two examples are given to show the value of reasonable reduction in sectional widths.

1. INDIAN FALLS.—CORFU ROAD IN NEW YORK STATE

Length 1.85 miles

No change in profile.
No change in ratio of cut to fill.

Original Design	Revised Design
Width of macadam, 14′.	Width of macadam, 14′.
Width of section, 30′.	Width of section, 24′.
Depth of ditch, 18″.	Depth of ditch, 14″.
Original estimated excavation, 7500 cu. yd.	Revised estimated excavation 5200 cu. yd.

This change in section alone resulted in a saving of 2300 cu. yd. excavation, or at a rate of 1240 cu. yd. per mile, or, in money, about $1200 per mile with excavation at $1 per cubic yard.

2. PITTSFORD.—NORTH HENRIETTA ROAD IN NEW YORK STATE

Length 2.67 miles

Original Design	Revised Design
Width of section, 30′.	Width of section, 24′.
Depth of ditch, 18″.	Depth of ditch, 12 to 14″.
Ratio of cut to fill, 1.35%.	Ratio of cut to fill, 1.25%.
Maximum grade, 5.0%.	Maximum grade, 5.0%.
Profile, designed with straight instead of rolling grades and tangents of 100′ between vertical curves.	Profile, rolling grade and reverse vertical curves used.
Original estimated excavation, 11,450 cu. yd.	Revised estimated excavation 6620 cu. yd.

A saving of 4820 cu. yd., 1800 cu. yd. per mile, or, in money, approximately $1800 per mile.

The revised design on this road is a good example of what can be saved by the use of a section that fits the conditions, a rolling grade, and a ratio of cut to fill that experience has shown to be sufficient.

Item 6. Stable Cut and Fill Slopes.—Economy of design and maintenance are affected by the selection of reasonably stable slopes. For the class of grading usually encountered on roads built in ordinary topography their effect on construction cost is not great and they do not generally receive much attention, but for mountain roads, cut and fill slopes are an important consideration in the design and their effects on cost are worth considering.

Table 128 (p. 786), shows the effect in detail of various cut and fill slopes on yardage of the ordinary side-hill mountain-road sections. To illustrate the point one typical case for, say, an ordinary double-track section (S-14), Table 128, will be quoted.

Natural ground surface cross-slope, in degrees	Approximate yardage per mile		
	Cut slope $1\frac{1}{2}:1$, fill $1\frac{1}{2}:1$, in cubic yards	Cut $1\frac{1}{4}:1$, fill $1\frac{1}{2}:1$, in cubic yards	Cut $1:1$, fill $1\frac{1}{2}:1$, in cubic yards
5	1,100	950	900
10	2,200	2,000	1,900
15	4,000	3,600	3,300
20	7,900	7,000	6,100
25	.....	11,700	10,200
30	.....	......	19,600

Occasional slides cannot be avoided, but continual slipping shows poor design and makes both the maintenance costly and travel dangerous.

Stable slopes vary for different materials and for the same material under different climatic conditions. A combination of moisture and frost requires the flattest slopes for ordinary soils. On account of the great variety of circumstances affecting the design, no hard-and-fast rules can be laid down, but the following table, based on railroad and highway practice, indicates the slopes that are generally used. In this table and throughout the text slopes are referred to as $1\frac{1}{2}:1$, etc., meaning $1\frac{1}{2}$ horizontal to 1 vertical. In some of the state standard illustrations, however, slopes are shown as 1 on $1\frac{1}{2}$, meaning 1 vertical on $1\frac{1}{2}$ horizontal. It is unfortunate that an engineering requirement is expressed by two different methods in such a conflicting order, and care must be taken to understand which expression is used.

TABLE 40.—STABLE CUT AND FILL SLOPES

Material	Climatic conditions					
	Combined rain and heavy frost		Rain but not much frost		Arid regions, not much frost	
	Cut	Fill	Cut	Fill	Cut	Fill
Sand	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$2:1$	$2:1$
	$2:1$	$2:1$	$2:1$	$2:1$	$4:1$	$4:1$
Gravel	.....	.....	$1\frac{1}{2}:1$	.....	$1\frac{1}{2}:1$	
	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$1:1$	$1\frac{1}{2}:1$	$1:1$	$1\frac{1}{2}:1$
Loam	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$1\frac{1}{4}:1$	$1\frac{1}{2}:1$
Clay	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	.....	$1\frac{1}{2}:1$	$1:1$	$1\frac{1}{2}:1$
	$2:1$	$4:1$	$1:1$	$3:1$	$\frac{3}{4}:1$	
Boulders and earth	$1\frac{1}{2}:1$	$1\frac{1}{2}:1$	$1:1$	$1\frac{1}{2}:1$	$1:1$	$1\frac{1}{2}:1$
Large rock slabs extending back into hill and earth	$1:1$	$1\frac{1}{2}:1$	$\frac{3}{4}:1$	$1\frac{1}{2}:1$	$\frac{3}{4}:1$	$1\frac{1}{2}:1$
Disintegrated rock and shale	$\frac{1}{2}:1$	$1\frac{1}{2}:1$	$\frac{1}{2}:1$	$1\frac{1}{2}:1$	$\frac{1}{2}:1$	$1\frac{1}{4}:1$
Solid rock	$\frac{1}{4}:1$	$1:1$	$\frac{1}{4}:1$	$1:1$	$\frac{1}{4}:1$	$1:1$

Examples of Typical Sections High Class Roads.—The following typical sections give an idea of the trend of present practice:

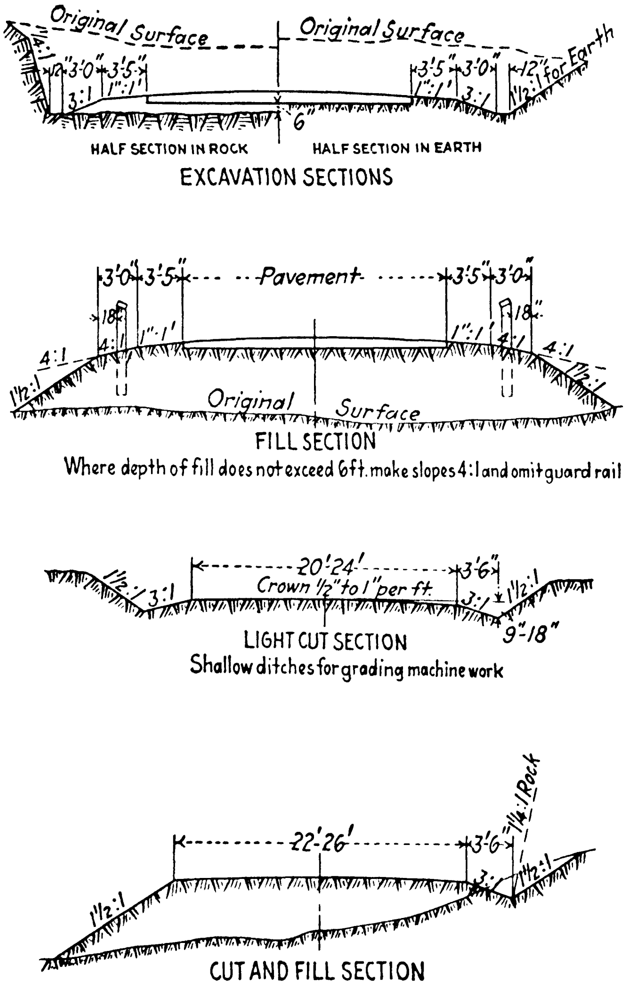


FIG. 28.—Earth road sections. U. S. Bureau Public Roads (1925).

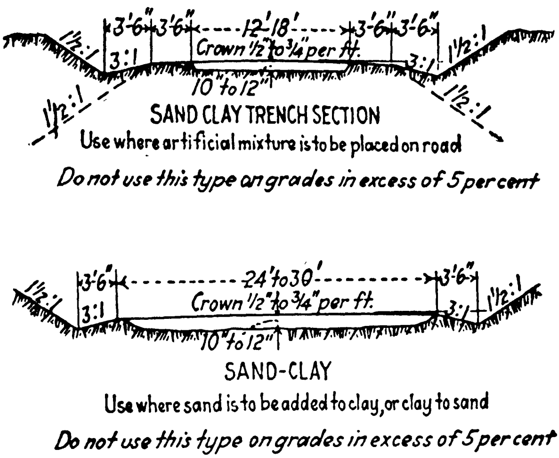


FIG. 28.—Sand clay roads.

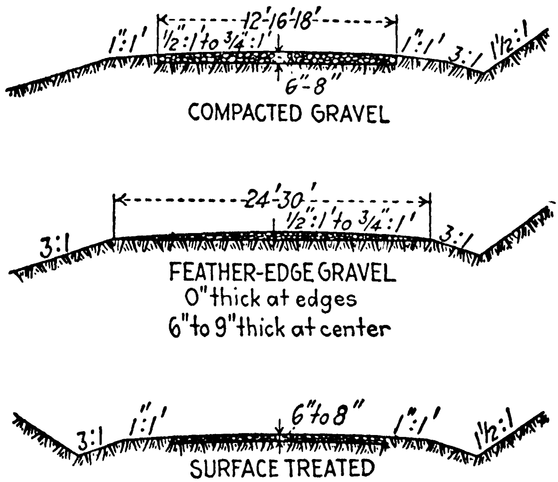


FIG. 28.—Gravel roads. U. S. Bureau Public Roads (1925).

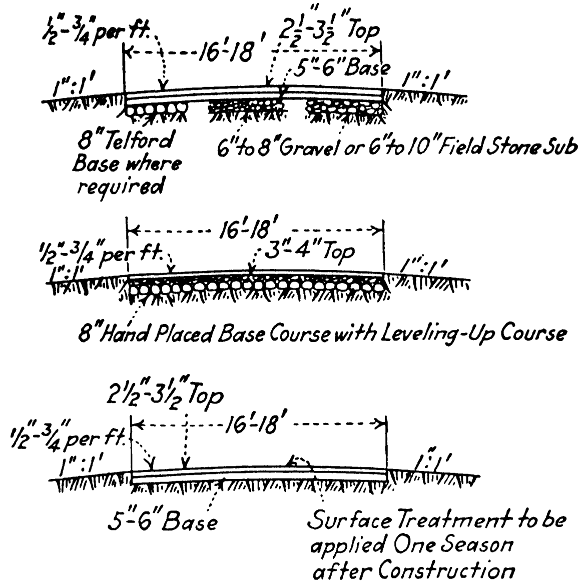


FIG. 28.—Waterbound macadam. U. S. Bureau Public Roads (1925).

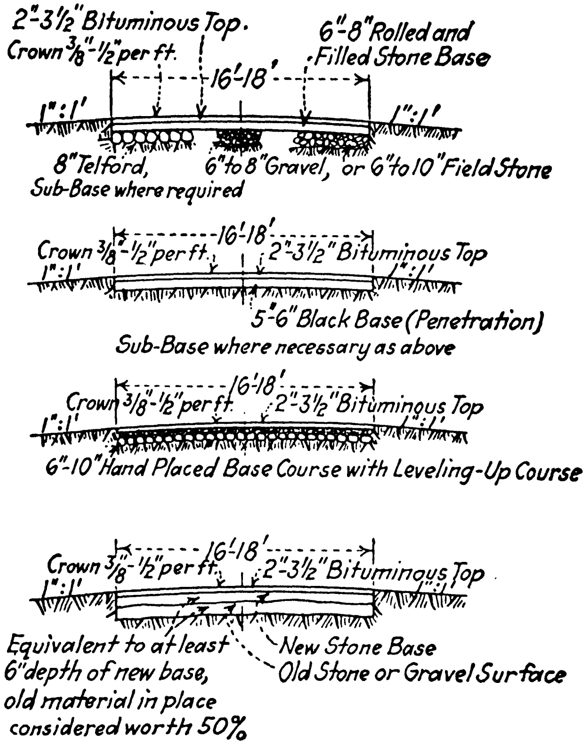


FIG. 28.—Bituminous macadam. U. S. Bureau Public Roads (1925).

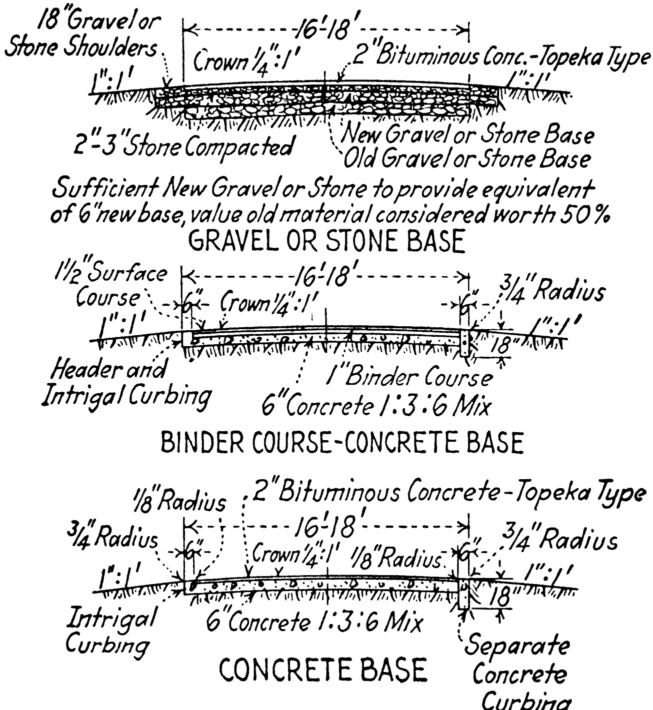


FIG. 28.—Bituminous concrete. U. S. Bureau Public Roads (1925).

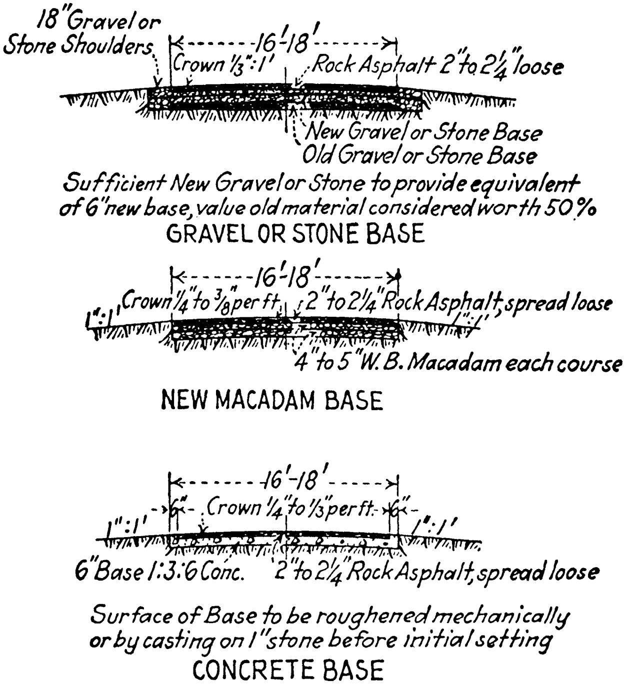


FIG. 28.—Rock asphalt. U. S. Bureau Public Roads (1925).

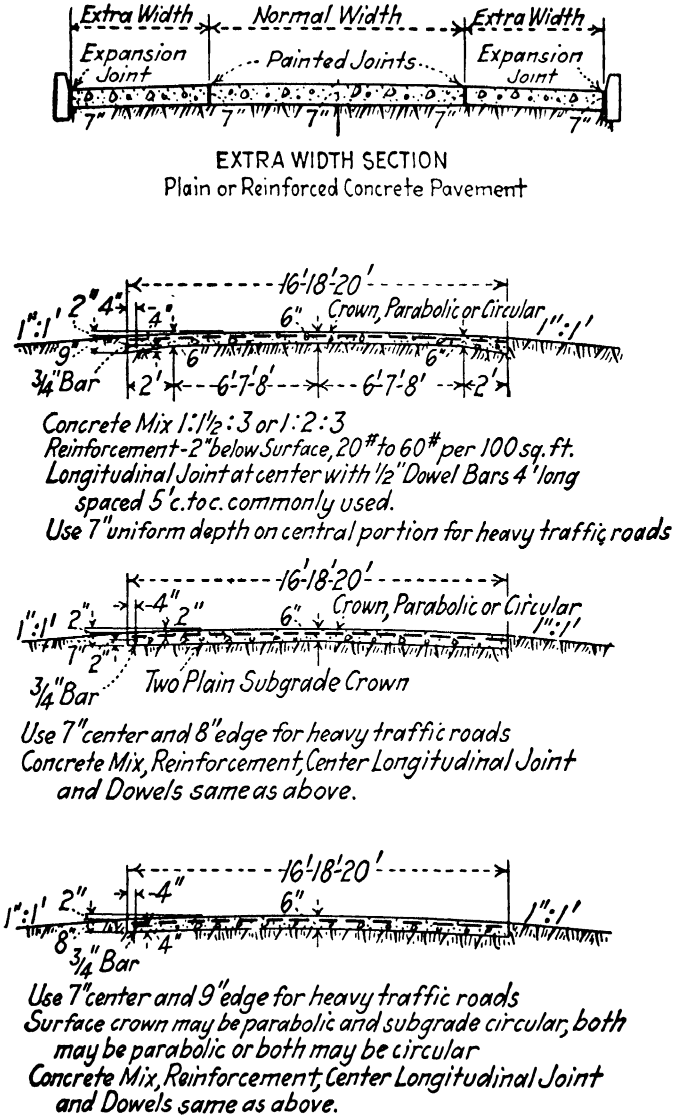


FIG. 28.—Cement concrete. U. S. Bureau Public Roads (1925).

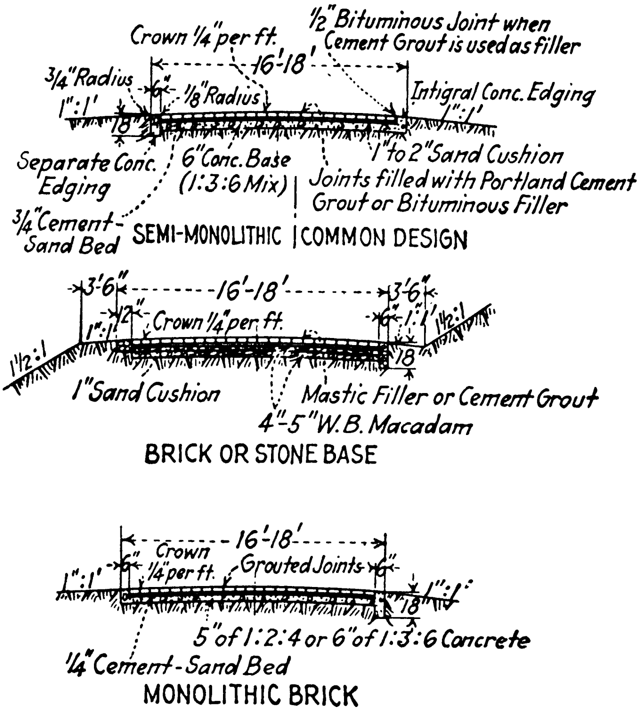


FIG. 28.—Brick. U. S. Bureau Public Roads (1925).

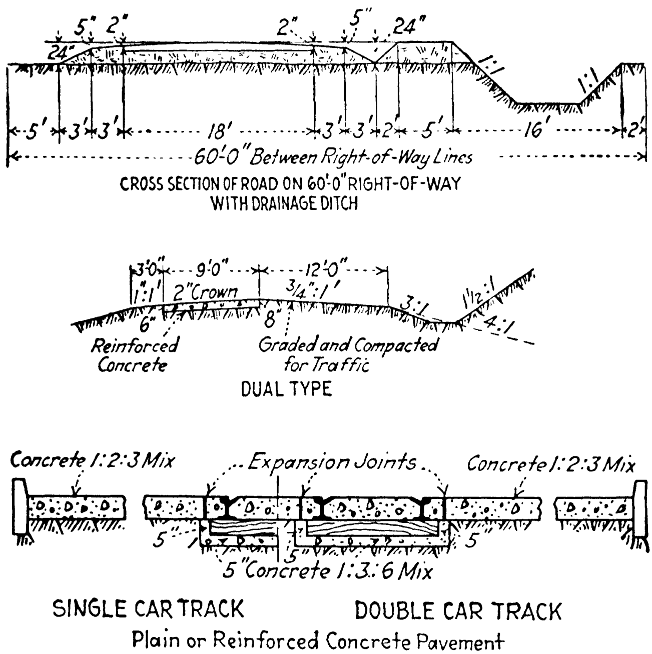


FIG. 28.—Miscellaneous sections. U. S. Bureau Public Roads (1925).

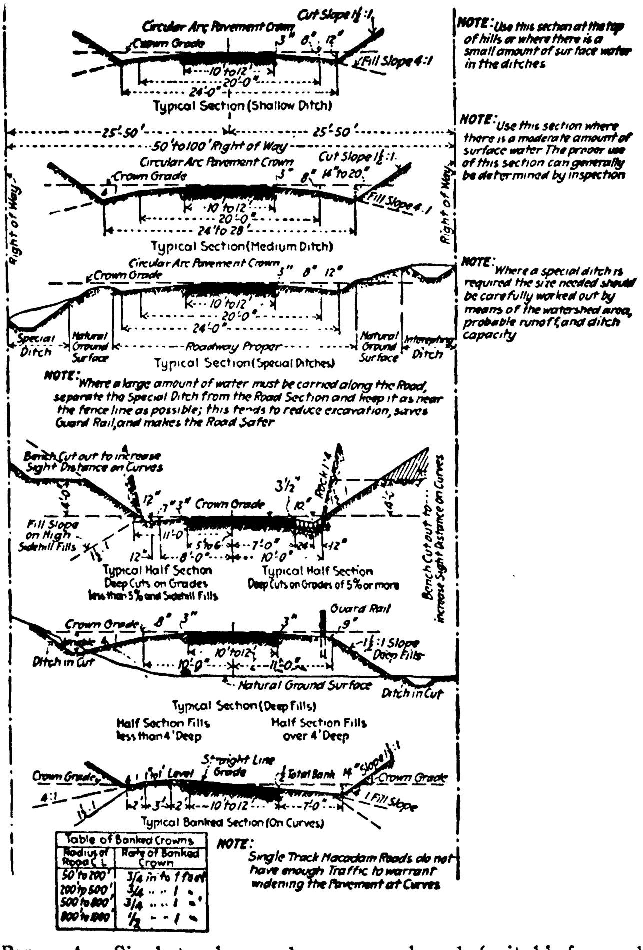


FIG. 29A.—Single track macadam or gravel roads (suitable for roads carrying up to about 300 vehicles per day). Typical sections.

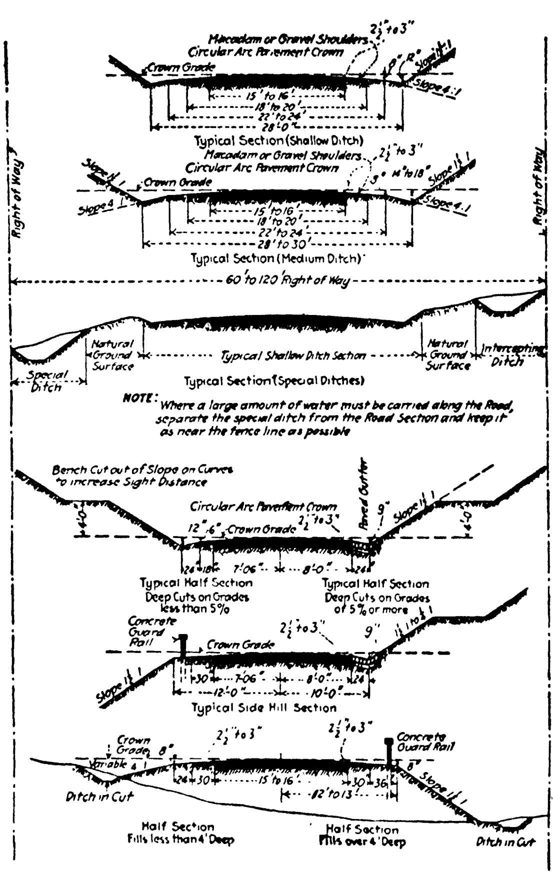
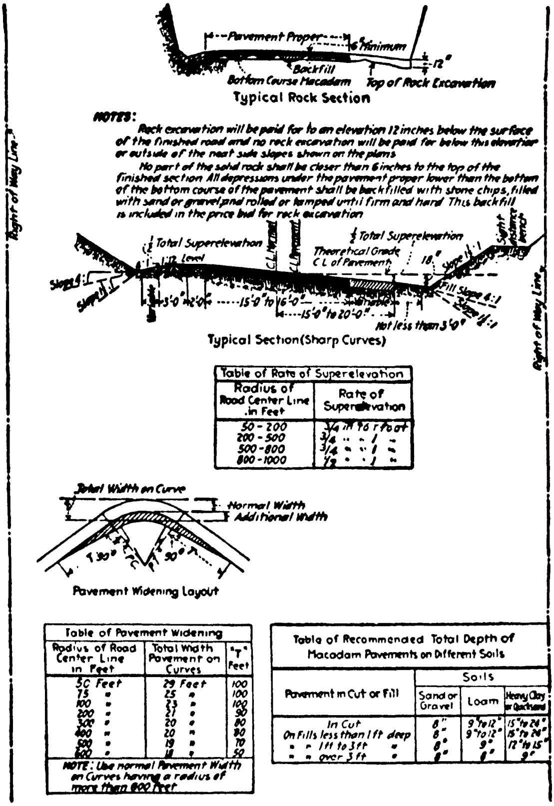


FIG. 29*B*.—Double track macadam roads. Suitable for local service or secondary State Roads carrying from 300 to 2000 vehicles per day (12 hour count in summer). Typical sections.

NOTE.—Use 7 ft. shoulder for traffic over 1500 daily average to permit parking off the pavement.



NOTES:

Rock excavation will be paid for to an elevation 12 inches below the surface of the finished road and no rock excavation will be paid for below this elevation or outside of the neat side slopes shown on the plans.

No part of the solid rock shall be closer than 6 inches to the top of the finished section. All depressions under the pavement proper lower than the bottom of the bottom course of the pavement shall be backfilled with stone chips, filled with sand or gravel and rolled or tamped until firm and hard. This backfill is included in the price bid for rock excavation.

Table of Rate of Superelevation

Radius of Road Center Line in Feet	Rate of Superelevation
50 - 200	3/4 in to 1 foot
200 - 500	3/4 " " 1 "
500 - 800	3/4 " " 1 "
800 - 1000	1/2 " " 1 "

Table of Pavement Widening

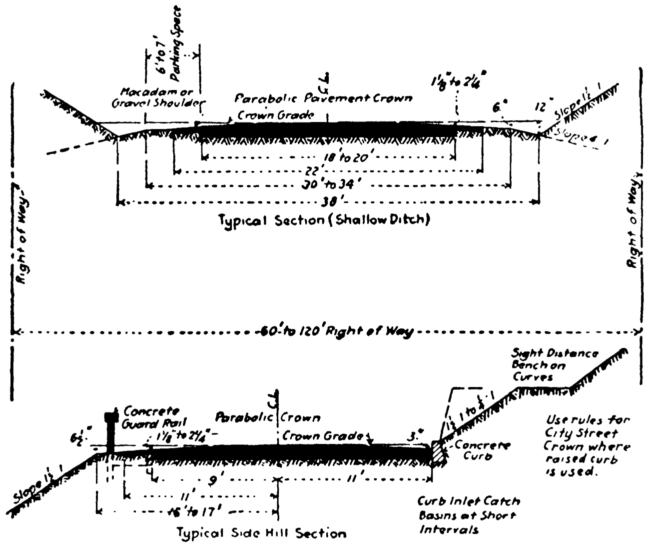
Radius of Road Center Line in Feet	Total Width Pavement on Curves	"T" Feet
50 Feet	29 Feet	100
75 "	25 "	100
100 "	23 "	100
200 "	21 "	90
300 "	20 "	80
400 "	20 "	80
500 "	19 "	70
600 "	18 "	50

NOTE: Use normal Pavement Width on Curves having a radius of more than 600 Feet

Table of Recommended Total Depth of Macadam Pavements on Different Soils

Pavement in Cut or Fill	Soils: Sand or Gravel	Loam	Heavy Clay or Quicksand
In Cut	8"	9" to 12"	15" to 24"
On Fills less than 1 ft deep	8"	9" to 12"	15" to 24"
" " 1 ft to 3 ft "	8"	9"	12" to 15"
" " over 3 ft "	8"	8"	9"

FIG. 29B.—Double track macadam roads. Typical sections.



TYPICAL MEDIUM DITCH SECTION
" SPECIAL " "
" FILL SECTION
" BANKED "
} Similar to Fig. 29*B* except that the Overall Width dimensions are 6′ to 8′ greater than given in Fig. 29*B*

ROCK EXCAVATION: Method of Payment similar to Fig. 29*B* except that none of the Solid Rock shall project above the bottom of the Pavement Base

NOTE: For thickness of Pavements and Design of Pavement Base and Surfacing See Chapter VI

FIG. 29*C*.—Special sections rigid pavement roads (roads carrying over 2000 vehicles per day. Typical sections.

STANDARD GRADING SECTION, ILLUSTRATING METHOD OF DESIGNATING ROADWAY WIDTH

Standard roadway width to be designated as width in thorough cut, dimension "A" in the table. Reference to a standard roadway width will imply the corresponding dimensions as shown in the table.

STANDARD ROADWAY WIDTHS	CUT A	SIDE HILL B	FILL C	1/2 A	1/2 C
20'	20'	21'	22'	10'	11'
24'	24'	25'	26'	12'	13'
28'	28'	29'	30'	14'	15'
30'	30'	31'	32'	15'	16'
40'	40'	41'	42'	20'	21'

FIG. 28A.—California standard road sections (1925). (State roads.)

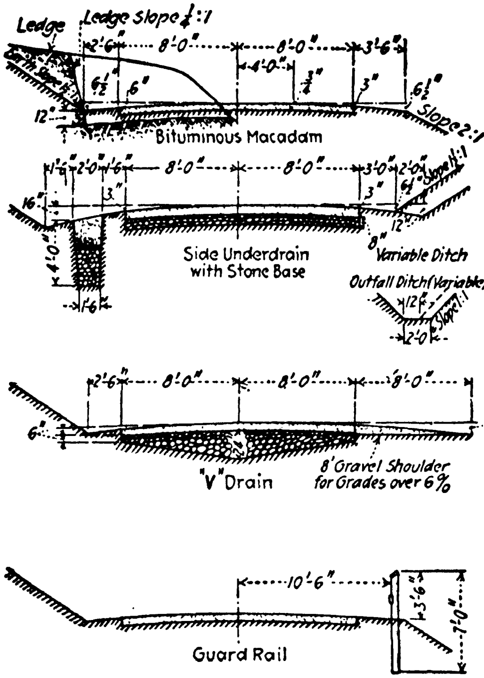


FIG. 28*B*.—State of Maine standards (1925).

MOUNTAIN-ROAD SECTIONS

Discussion.—The desirable requirements for mountain-road sections are the same as for roads previously discussed, but on steep side-hill work the width of grading used for ordinary topography would be prohibitive in cost. As most of these roads are natural-soil roads, the crown is the only element of the section not covered in the previous discussion. For the gravel or stony material usually encountered $\frac{1}{3}''$ to $1'$ is generally satisfactory. For sand or heavy soils $\frac{1}{2}''$ or $\frac{3}{4}''$ to $1'$ is better practice. The old idea that crown should be increased on steep grades has been abandoned for, while that expedient undoubtedly helped the drainage, it caused more inconvenience to traffic than it was worth. In many cases present practice decreases the crown on steep grades to give better vehicle control. Crowns on mountain roads are also affected by the absence of guard rail or other safety provisions. The ordinary symmetrical crown is used where wall or guard rail protects the dangerous out-

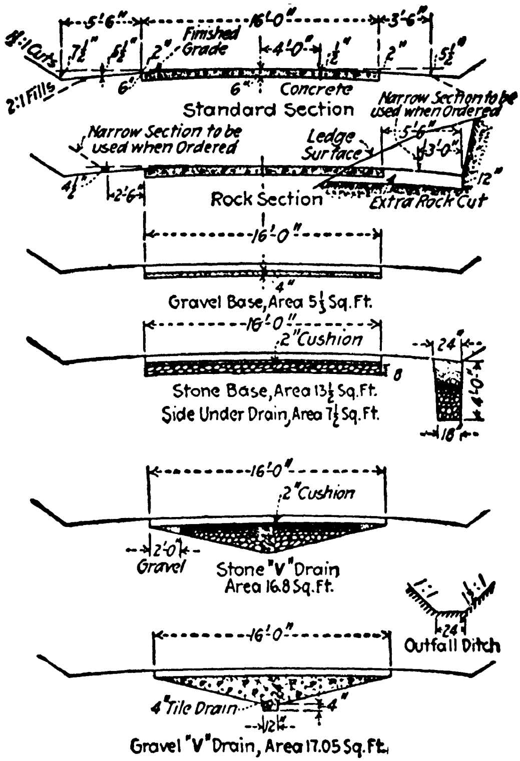


FIG. 28*B*.—(*Continued.*)

side slope, but on many roads so much rail would be needed that it is prohibitive in cost, and where it cannot be used the road is tipped one way in a continuous slant toward the hill so that if a machine skids it will slide in against the cut slope. This kind of section is not so comfortable to ride as the ordinary crown, but if the surface is at all greasy the element of increased safety outweighs any minor inconvenience of side tilt.

Effect of Width on Cost.—The width of section has more effect on cost than any other part of the design. On a new side-hill location the relation of width to cost can be roughly established. It will, of course, vary for different side slopes of the hill and different cut slopes of the excavation, but the relation will be approximately as follows, for balanced sections (Table 128, p. 786).

EXCAVATION PER MILE. 25° SIDE-HILL SLOPE

1: 1 slope in cut. 1½ : 1 slope in fill

(S-8) 10′ width (ditch to outside of shoulder) 4,300 cu. yd. per mile
(S-10) 12′ width (ditch to outside of shoulder) 6,100 cu. yd. per mile
(S-14) 16′ width (ditch to outside of shoulder) 10,200 cu. yd. per mile
(S-16) 18′ width (ditch to outside of shoulder) 12,800 cu. yd. per mile
(S-18) 20′ width (ditch to outside of shoulder) 15,400 cu. yd. per mile

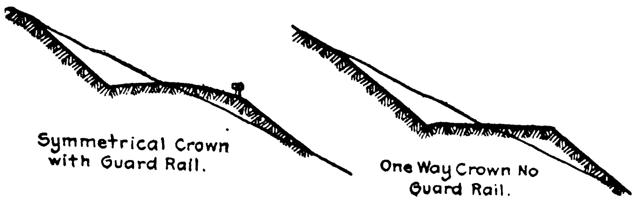


FIG. 30.

In general, a 20′ width requires about three and one-half times as much excavation as a 10′ width. The relative cost of different widths is also affected by the amount of rock excavation, which is generally much greater for the wider widths. This depends on the depth of soil overlying the rock. This element affects the cost so much that in certain cases it has been found cheaper to build two separate single-track roads for short distances rather than one double-track highway.

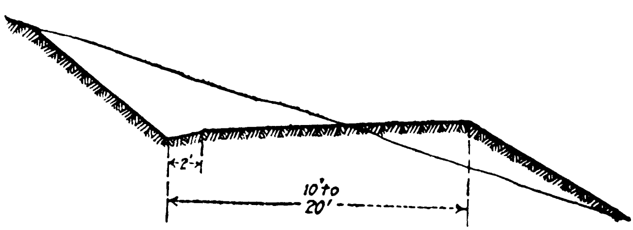


FIG. 31.

Mountain roads are classed roughly as double track or single track, meaning the same as for railroad work, a double line of traffic or a single line with turnouts to allow passing. As each foot of extra width is costly, it is important to determine the minimum width of grading that will serve the purpose for these two classifications.

Minimum Width Side-hill Section.—If the roadbed is benched out of solid rock, a narrower width will serve as the entire width is firm and stable. If the section is a balanced section part in cut and part in fill, it must be wider, as embankments on steep slopes are liable to settle, slide, or wash out and it is not safe to drive so closely to the edge as in the first case. The amount of road "in solid" is therefore the prime requisite and "feet in solid" is often used as the specification for contract road jobs where engineering design

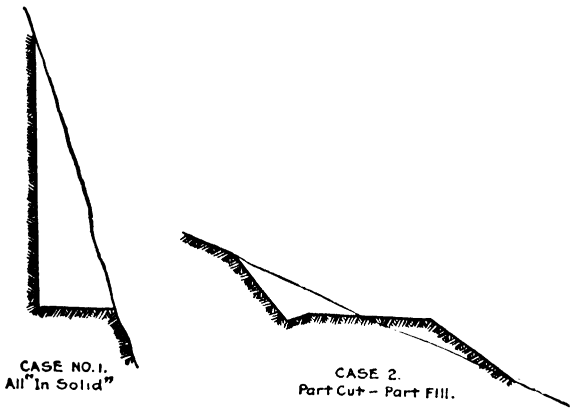


FIG. 32.

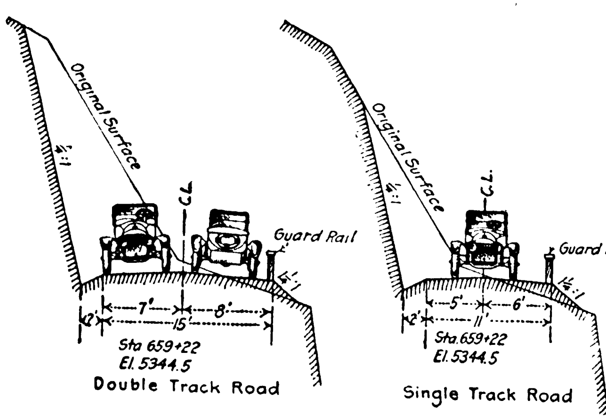


FIG. 33.—Single and double track roads.

is not used. Present practice favors a minimum single track, total grading width of 10′ in rock or where the outer embankment is sustained by a retaining wall and a total width of 12′ for the ordinary balanced section in earth. Balanced sections are generally used up to 30° side slopes and beyond that toe walls or retaining walls are necessary for earth sections. For a 30° side slope a total grading width of 12′ results in approximately 7 to 8′ in solid cut. A double-track section requires a minimum total grading width of 14′ in rock or wall sections and 16′ in balanced earth section, which gives approximately 10′ in solid. These same limiting widths apply to turnout sections on single-track roads. Where guard rail is used 1′ should be added to these widths. These widths are, however, very skimpy, and if the money is available at least 2′ additional should be used.

Turnouts.—On single-track roads turnouts are constructed at sufficiently frequent intervals so that drivers can see between them and there will be no danger of meeting at impassable spots. This generally requires from 5 to 10 to the mile. The minimum satisfactory length of turnout is about 60′ and the grade should be as easy as possible at these points.

Fill Sections.—Through fill sections must be constructed wider than side-hill sections, as the sides are bound to slough off under weather action and all the elements of wear tend to decrease the width; 14′ is considered the minimum width for a single-track road and 20′ the minimum for a double-track. A symmetrical crown is advisable on fills even on curves. Where guard rail is used, increase these widths 2′. These sections occur on only a small per cent of the length of mountain roads.

Through Cut Sections.—These sections are rare in occurrence; the minimum width, ditch to ditch, for single-track roads can be considered as 12′ and for double-track 18′. The use of minimum widths for either through cut or fill sections on mountain roads has small effect on cost, and for that reason more liberality in their widths is allowable.

Turnpike Sections.—Where the natural ground cross-slope is less than 5°, turnpiking is the usual construction and the difference in cost of a single or double track is so small that it is not worth considering. For this class of section a minimum of 22′ between ditches will apply to any road, and a width of 24′ is generally used.

Selection of Section.—Plate 35 (pp. 166 to 171), illustrates typical mountain-road sections, pioneer districts.

The turnpike section is used up to natural ground side slopes of 5° for continuous balanced work.

The side-hill sections are used above 5° for continuous balanced work. The one-way crown is used on all single-track side-hill sections where guard rail is lacking. The one-way crown is used on unprotected double-track roads where the side slope is greater than 15°. The symmetrical crown is used on protected double-track roads and on unprotected sections where the side slope is less than 15°.

Through cut and fill sections are used where required by the profile.

Superelevation is used on curves, but rarely on high through fills. The ditch on the upper side of a superelevated through cut section can be omitted if the cut is short.

Cut and fill slopes depend on the natural material and climate and were discussed on page 144. There is too much tendency to use steep slopes to save on construction cost, although excessively flat slopes are not necessary or advised, it being cheaper to take care of minor slides by maintenance (for effect of cut slopes, see Table 128, p. 786).

Wall Sections.—These sections are used where the natural hill slope is practically as steep or steeper than the stable embankment slope. Toe or retaining walls are necessary for earth embankments where the natural slope exceeds approximately 30° and for rock fills where the natural slope exceeds approximately 40°. Wall details are described in Chapters X and XII. Surcharged breast walls are to be avoided if possible.

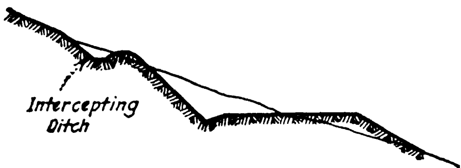


FIG. 34.

Intercepting Ditches.—Where considerable water runs down the uphill slope, intercepting ditches are used to protect the cut slope and relieve the road ditch of excess water. These ditches discharge to the nearest cross-culvert and are an important part of the design.

Bench Sections.—Bench sections are used in rock ledge work (see sec. S-10, Plate 35, and Table 128, p. 786).

STREET SECTIONS

Widths.—This book is primarily concerned with the design of rural highways, but in connection with rural improved highways it is often necessary to design short stretches of village or city street pavements as connecting links on such systems, and proper widths of pavements and sidewalks on streets become an element in the designs to be handled. City-street layouts and widths are big subjects, covering a wide range from unimportant resident streets to pleasure boulevards and congested main streets. The subject is well covered by works by Robinson, Lewis, Blanchard, Agg, etc., and for special problems readers are referred to such works.

Connecting links in state highway systems through villages generally refer to main residential or business streets of villages or small cities. For these conditions, central parking spaces are rarely desirable, and the layout is the common one of roadway with side lawns or wide sidewalks from the curb to the street line.

For residential streets where occasional parking parallel to the curb prevails, a width of paved street of 30 to 32′ between curbs is a satisfactory minimum on state route connecting links through

(text continued on page 174.)

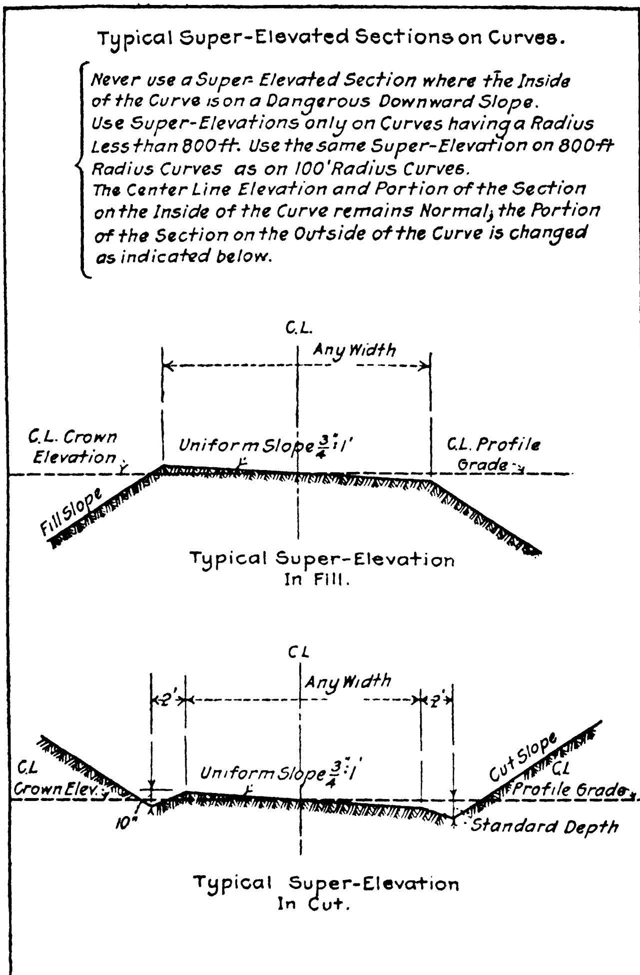


FIG. 35.—Pioneer mountain roads.

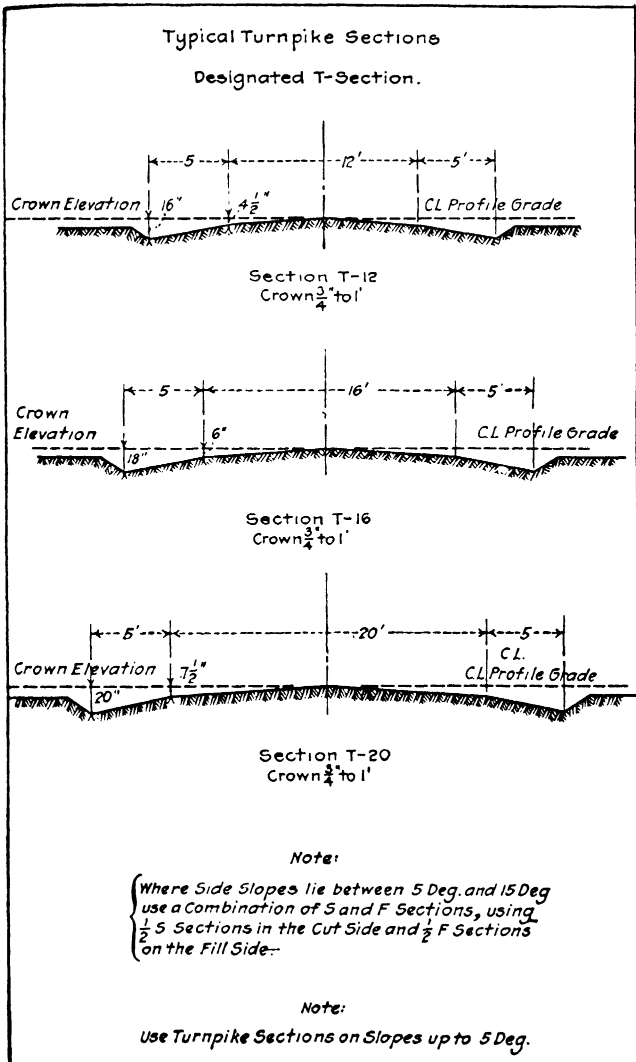


FIG. 35.—Pioneer mountain roads (*continued*).

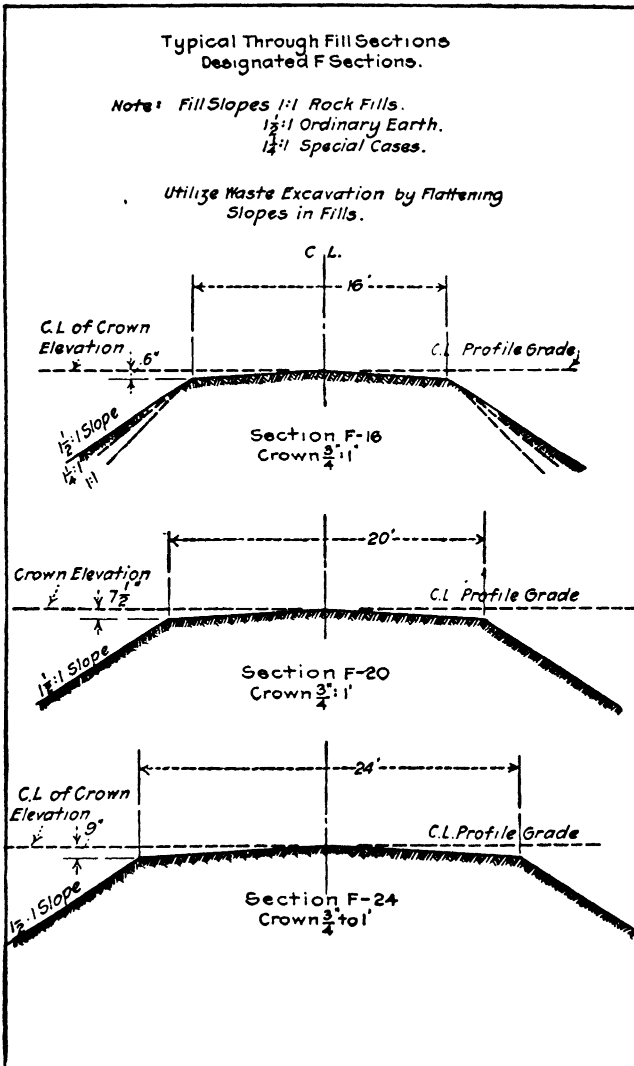


FIG. 35.—Pioneer mountain roads (*continued*).

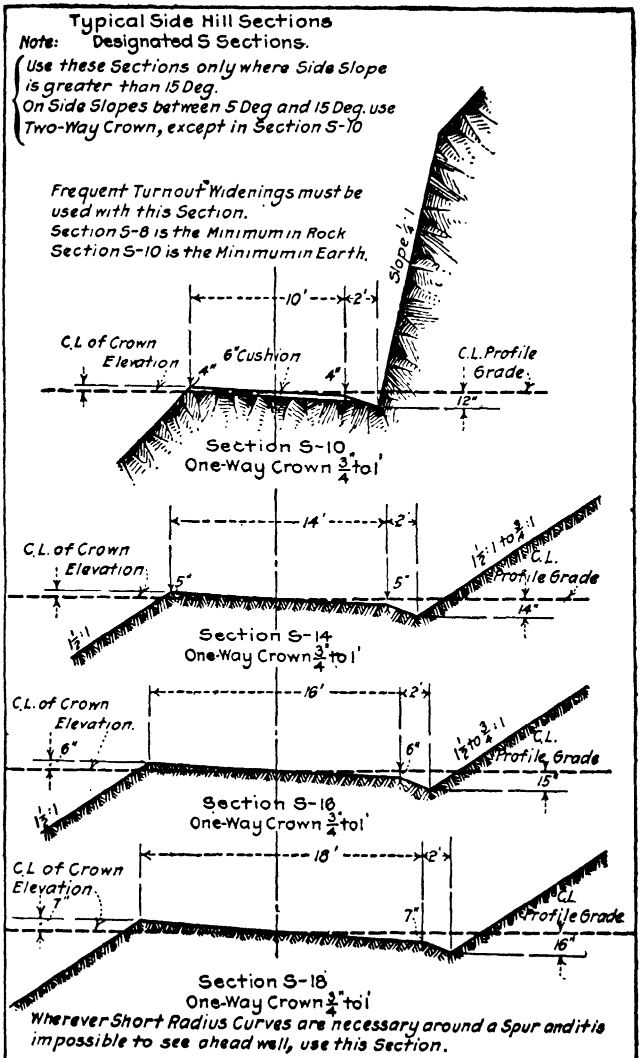


FIG. 35.—Pioneer mountain roads (*continued*).

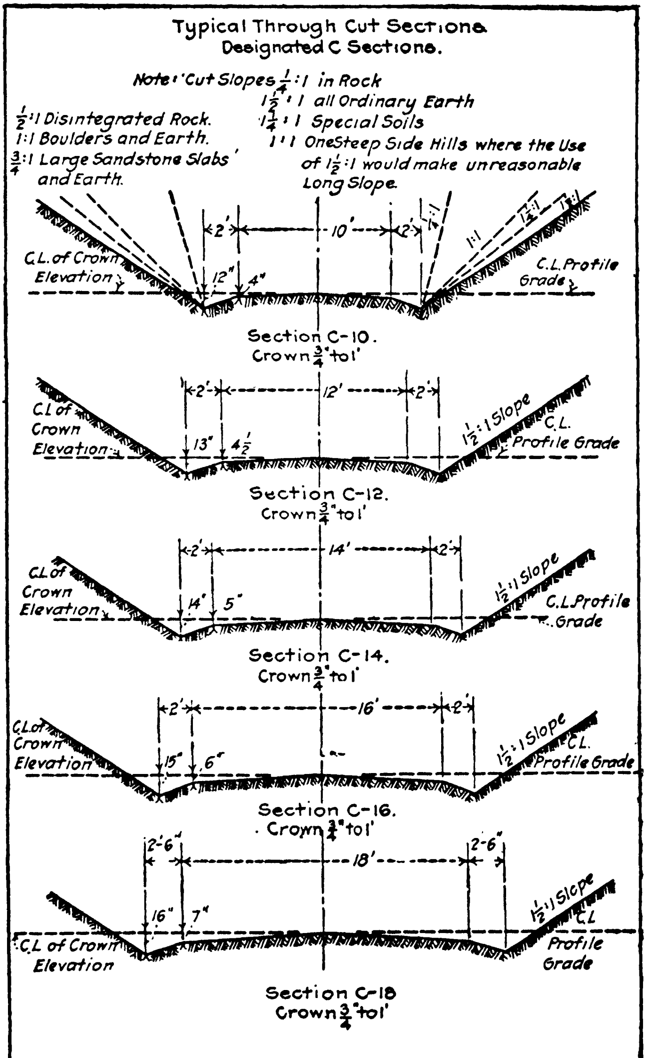


Fig. 35.—Pioneer mountain roads (*continued*).

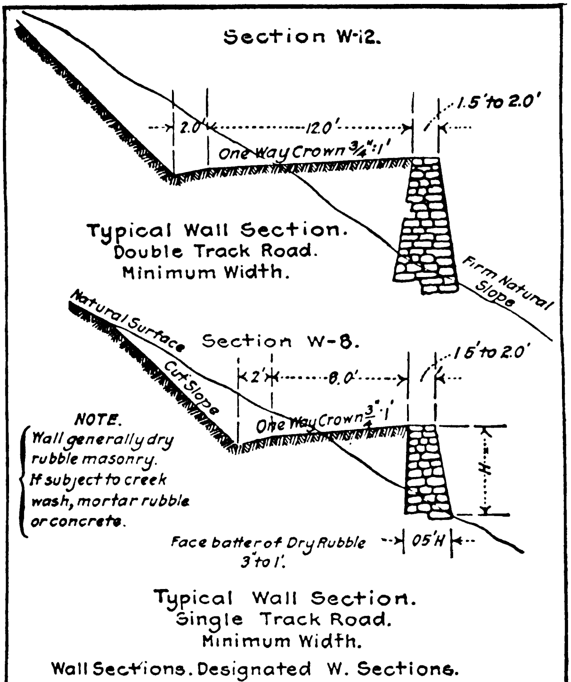


FIG. 35.—Pioneer mountain roads (*continued*).

STREET WIDTHS

Chicago Regional Planning Commission recommends widths of paving as follows:

	Feet
A. Lane for moving traffic	10
B. Lane for parking parallel with curb	8
C. Lane for parking at angle with curb	18

For single-family residential streets 66-ft. right of way and 26-ft. pavement width.

For main residential streets 66-ft. right of way and 36-ft. paving width (two moving lanes and two parking lanes).

For business streets 80- and 100-ft. right of way with 56- or 76-ft. paving for four or six moving lanes and two parking lanes.

Author's Note: For a moving lane where vehicles are largely passenger autos 9 ft. is ample; 10 ft. is required for trucks and busses.

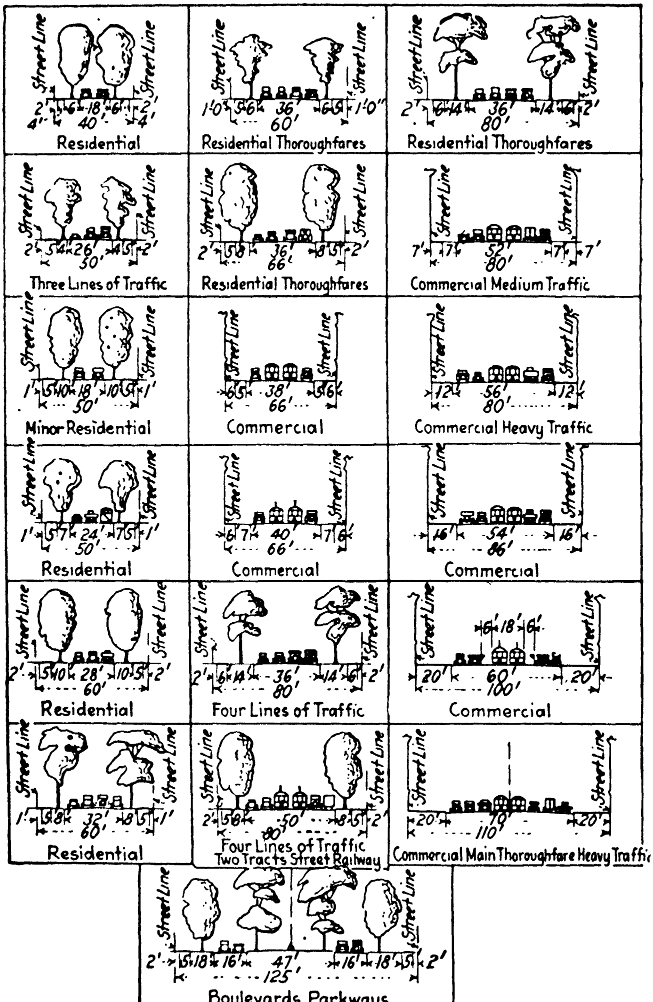


FIG. 36.—Street widths. City Planning Commission, Rochester, N. Y., 1926.

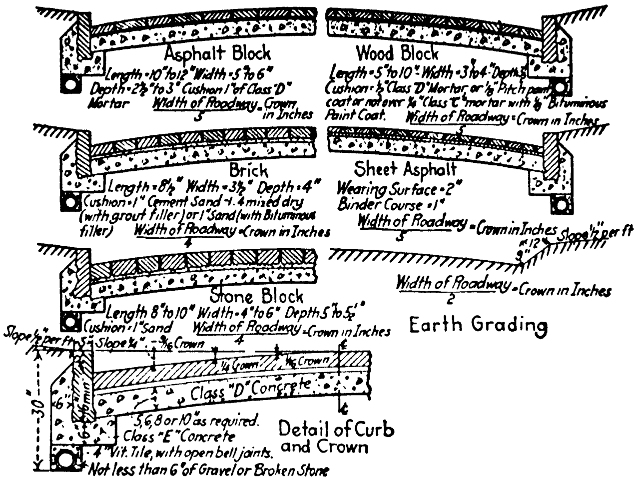
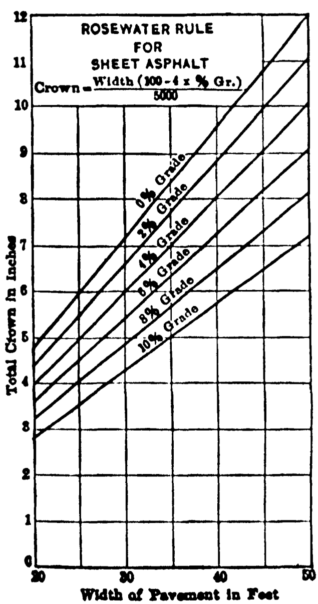
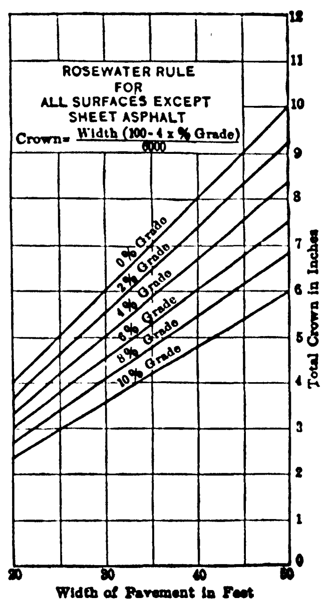


FIG. 37.—Rochester street crowns





villages where there are no street car tracks. Where double car tracks occur it is customary to increase the minimum to 36′, although this width does not give as great freedom of movement and a width of 50′ is advisable if the total width between street lines

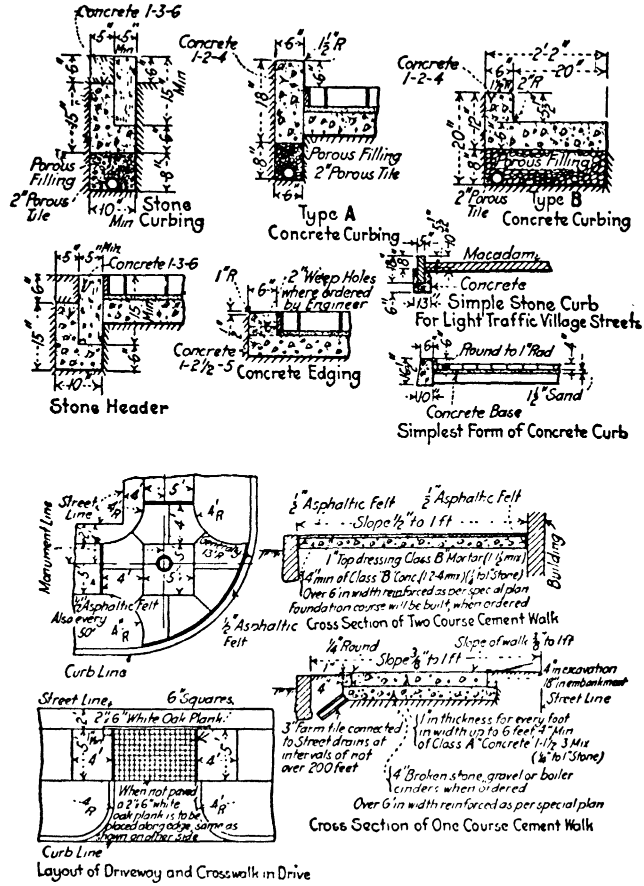


FIG. 39.—Sidewalk and curb details.

will permit. Sidewalks 5′ wide with a variable width of ornamental grass, tree, or shrubbery parking with a minimum cross-slope of ¼ to ⅜″ per foot will serve very well. Excessive pavement widths are to be avoided on residential streets and wide side parking adds to the pleasing appearance of the street.

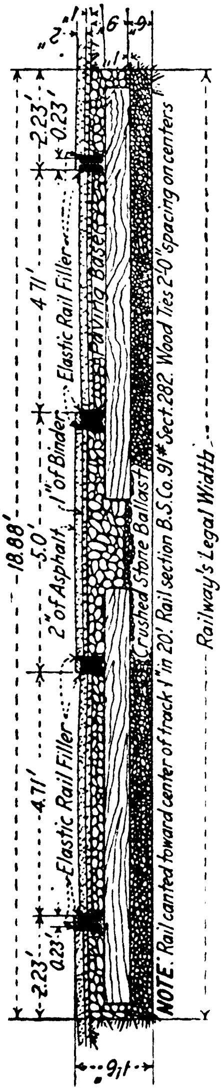
In business sections a minimum width of 50 and preferably 60′ between curbs is desirable where there are no car tracks. Sidewalks from curb building line should be at least 12′ if the total width of the street permits. For wide business streets it is rarely necessary to make the sidewalk more than 18′, the pavement taking up the balance of the space. Where street car tracks occur, it is desirable to make the pavement as wide as possible without reducing the sidewalk width below the 12′ limit.

FIG. 40.—Typical car track sections (State of New Jersey, 1922).

Figure 36 shows good typical street-paving widths.

Crowns.—On curbed streets, the crown should be more than on rural highways to concentrate flow into a narrow gutter area, and also because a flat crown does not look so well as a moderately high crown. Figures 37 and 38 show good standard practice which looks well, rides easily and drains well. Circular arc or parabolic crown curves are recommended (see p. 956 for method of figuring crown ordinates).

Sidewalks.—Figure 39 shows the details of satisfactory sidewalk construction. Cross-slopes of ¼″ per foot are satisfactory for concrete walks and ½″ per foot for brick walks.



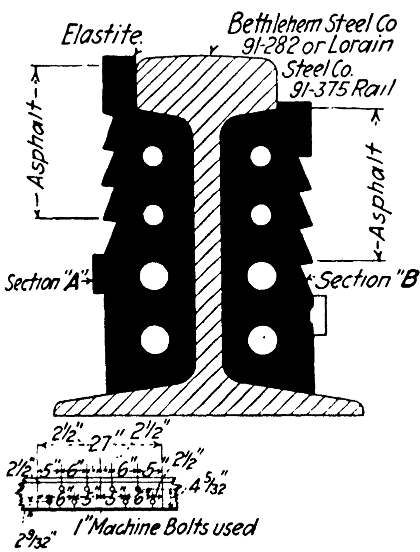


FIG. 41.—Street car track details, Rochester, N. Y. (1926).

Car Tracks.—Figures 40 and 41 show good typical car track construction.

Incidentals.—For details of curbs, catch basins, manholes, lighting posts, etc., see Chap. X (pp. 685 to 696).

RIGHT-OF-WAY AND CLEARING WIDTHS

The width of right of way is determined by required grading widths, by required clearing widths, by possible future widening of the grading, and by a minimum sight distance where buildings may be erected directly on the road boundary or where a heavy stand of brush or trees grows on the land back of the road boundary. While it is desirable to provide sufficient width for all the requirements of the future, the use of a needless width results in waste land which might better be utilized for farming or building purposes. There have been cases of rights of way 500′ wide in flat country, which are merely ridiculous.

The ordinary double-track improved road section varies from 24 to 36′ ditch to ditch. The cut and fill slopes back of the ditch

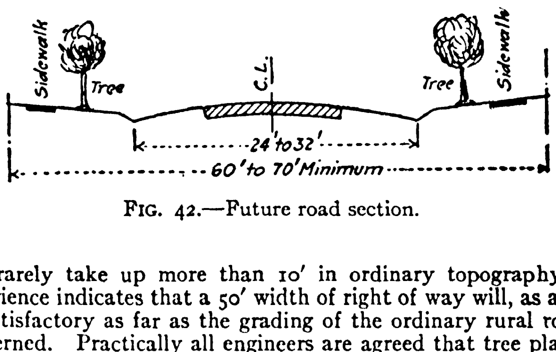


FIG. 42.—Future road section.

line rarely take up more than 10′ in ordinary topography and experience indicates that a 50′ width of right of way will, as a rule, be satisfactory as far as the grading of the ordinary rural road is concerned. Practically all engineers are agreed that tree planting and sidepaths for pedestrians are only a matter of time, and that an allowance for improvements of this kind is reasonable. Such an allowance would naturally increase the normal right-of-way width for the usual local service road to approximately 60′.

On state and national routes where four lines of traffic are anticipated a normal width of 80′ ought to serve satisfactorily except as modified for deep cuts and fills, sight distance on sharp curves, and clearing widths.

Modifications for deep cuts and high fills show up on the cross-sections. Modifications for sight distance can be worked up diagrammatically for each case, but in order to give some idea of the approximate increase in right-of-way widths for sharp curves the following tables are inserted.

TABLE 41.—TABLE OF DISTANCE BETWEEN CENTER LINE OF ROAD AND RIGHT-OF-WAY LINE ON THE INSIDE OF THE CURVE TO PERMIT CERTAIN SPECIFIED SIGHT DISTANCES, ASSUMING THAT THE LINE OF SIGHT IS NOT OBSTRUCTED WITHIN THE LIMITS OF THE RIGHT OF WAY AND THE CURVE IS LONGER THAN THE SIGHT DISTANCE REQUIRED

Road center-line radius, in feet	200′ sight distance	300′ sight distance	400′ sight distance	500′ sight distance
	Values given below are the distance from the road center line to the right of way on the inside of the curve to give the sight distance shown at the head of each column.			
100	100.0			
150	38.0	150.0		
200	26.8	64.3	200.0	
250	20.8	50.0	100.0	250.0
300	17.1	40.2	76.3	134.0
400	12.7	29.2	53.6	87.6
500	10.1	23.0	41.8	67.0

TABLE 42.—TABLE OF RADII OF CENTER LINE REQUIRED FOR DIFFERENT SIGHT DISTANCES AND DIFFERENT RIGHT-OF-WAY WIDTHS, ASSUMING THAT THE LINE OF SIGHT IS TANGENT TO THE RIGHT OF WAY LINE

This permits building being erected on the line. This table indicates minimum curvature for certain limiting right-of-way widths, metropolitan districts.

Total width of right of way in feet, center line of road located in center of right of way	200′ sight distance	300′ sight distance	400′ sight distance	500′ sight distance
	Values given below are the approximate radii in feet of the road center line to give the required sight distance.			
50	212	463	812	1262
60	182	390	682	1056
80	145	301	520	801
100	125	250	425	650

Modifications for clearing depend on the height and thickness of the growth. The object of clearing is, first, to remove growth within the slope lines; second, to provide a clear view; and, third, to clear sufficient width to allow the sun to reach the road, dry it out, and melt snow. This last depends a good deal on the direction in which the road is running, the altitude, and geographical location. It is entirely a matter of judgment, but should be liberal in the forest districts and ranges from 30′ in low growth to 150′ in adverse locations and high growth. In high altitudes roads are at their best closed in winter, and if careful location and liberal clearing will increase the length of the open season it is well worth while, as, in effect, it increases the usefulness of the road by 15 to 25%.

Recommended Practice.—All the evidence seems to indicate that the following normal right-of-way widths will be satisfactory, provided they are modified for unusual conditions of grading, sight distance, and clearing.

TABLE 43

	Main routes, in feet	Secondary roads, in feet	Local roads, in feet
Mountainous regions (cheap land)	150	100	100
Farming country (moderately cheap land)	100	70	50
Metropolitan districts (expensive land)	80	60	50

CHAPTER IV

DRAINAGE

CULVERTS, BRIDGES, DITCHES, UNDERDRAINS, AND STORM SEWERS

The problem of drainage may be divided into three parts:

1. Cross-drainage (pp. 180 to 333).
2. Longitudinal surface drainage (pp. 334 to 338).
3. Underdrainage (pp. 338 to 341).

Adequate drainage is essential to the success of road construction. As this element of design and construction is a fundamental permanent requirement, it is entitled to reasonable liberality in expenditure and to reasonable allowances for future flow and traffic conditions. Liberal waterway, safe roadway widths, and adequate strength of structures must be provided. Reasonable economy in drainage design is confined to the selection of the most economical type of structure which will serve present and possible future conditions. The selection of economic type depends on the topography of the stream crossing, height of banks, foundation conditions, length of span, proper use of piers, the necessity of future widening of the superstructure, the relative local costs of concrete or stone masonry and steel and the utilization of old structures either in whole or in part. Utilization of old structures very materially reduces bridge costs as discussed on pages 210 and 1013. A typical preliminary investigation report is given on page 327, supplemented by quick-estimating diagrams to illustrate economic selection of culverts and bridges. It is essential that each structure receive careful economic engineering design analysis. Stereotyped use of standards is to be avoided. A sliding-scale standardization of small culverts and bridge superstructures is convenient and applicable to most cases. *Bridge foundation design, however, cannot be economically standardized.*

CROSS-DRAINAGE

Cross-drainage includes culverts and bridges located at natural stream crossings, natural swales, artificial drainage or irrigation ditches, low points on the road profile, equalizing culverts where the road passes through a naturally depressed sump area, overflow culverts in flooded areas, and ditch relief culverts on long grades.

If the funds are limited, the cheaper types may be used, but all necessary structures must be built not only to protect the road but to establish a reasonable drainage scheme which is recognized and becomes fixed by usage as the country develops; it is very difficult to change surface drainage in well-settled districts without annoying and expensive lawsuits.

This chapter gives a short discussion of the essential elements of design, illustrated by standard current practice for culverts and small-span bridges. Long-span bridges are a specialized subject and the reader is referred to standard bridge books for data on their design and construction. Graphs (pp. 310 to 325) give sufficient data for preliminary estimates of cost and the selection of economic type for such bridges. The points to be considered in culvert and bridge design are:

a. Location of structure and channel improvements.
b. Waterway area and clearance for ice and débris.
c. Slope and elevation of culvert inverts.
d. Dead and live loadings.
e. Roadway widths and clearances.
f. Type of structures.
g. Protection from scour.

Location of Structure.—Poor location of structure is the most prevalent fault of the usual road drainage scheme. A good location fulfils the fundamental requirement of getting water across and away from the road as soon as possible. It also considers the desirability of a fairly uniform velocity of flow of the water in the channel and through the structure in order to minimize scour or silting up of the waterway. Sharp changes of direction in the flow, of water are undesirable, as they check velocity and produce ice or débris jams as well as tend to clog the channel with silt deposits. Comparatively inexpensive channel straightenings often materially reduce the spans and cost of bridges. *The necessity for careful studies of location and channel improvements is emphasized.*

Simple Illustrations of Culvert Locations. *Case I. Simple Right-angle Stream Crossing* (Fig. 43).—There is never any doubt in

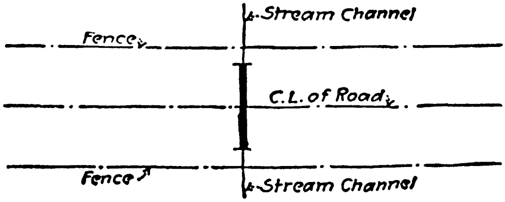


FIG. 43.—Case I. Culvert location.

this case. The structure is placed directly in the stream line and at right angles to the road center line.

Case II. Stream Crossing on Skew Angle (Fig. 44).—In a case of this kind it is desirable to place the culvert in line with the natural stream channel.

The right-angle location marked "poor" saves length of culvert, but generally requires four sharp changes in direction of flow which tend to check the velocity of flow and to produce scour and silting up at the angles. Considering maintenance cost, this type is generally poor economy unless the creek channel can be changed for some distance.

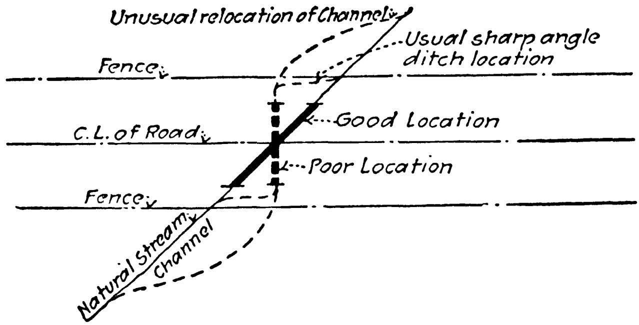


FIG. 44. Case II. Culvert location.

Case III. Where Stream Must Be Carried Along Road for Some Distance (Fig. 45).—The location marked "good" gets the water on to the low side of the road as soon as possible, minimizes sharp changes in the direction of flow, and is desirable unless houses or barns are located on the low side of the road between the points at which the stream strikes and leaves the road.

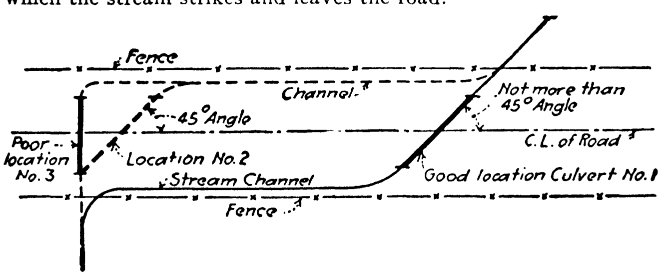


FIG. 45.—Case III. Culvert location.

Location 2 is desirable where houses are located on the low side of the road but not at the high side.

Location 3 is not desirable under any condition, as it checks flow and causes trouble by reducing the culvert capacity and encouraging scour and silting. In a number of cases locations of this nature have proved very unsatisfactory.

Case IV. Ditch Relief Culverts on Side-hill Location (Fig. 46).—Ditch relief culverts on side-hill locations are very desirable, as they minimize ditch scour. They are placed at any natural gully formation and on uniform slope formations are spaced from 300 to 500'.

The spacing between these ditch relief culverts on hill-side locations depends on the grade, soil, ditch lining, and width of section. A narrow 10′ mountain road requires more relief than a 20′ road in the same location, as even a small washout will put the narrower road out of commission, while a moderately bad ditch scour will not stop traffic in the second case. No set rules on spacing can be given, but current practice favors ditch relief culverts on

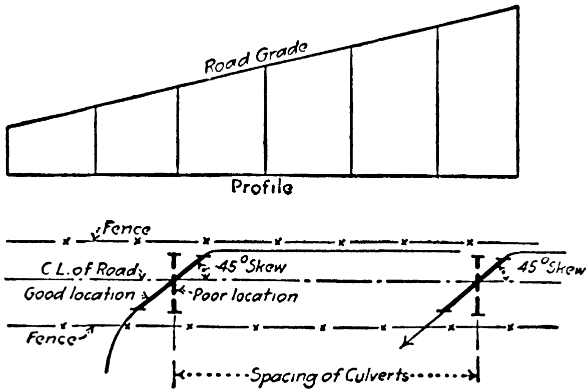


FIG. 46.—Case IV. Ditch relief culverts.

8% grades at intervals not exceeding 300′, and on 5% grades not exceeding 500′. If cobble gutter or concrete ditch lining is used, the distance can be materially increased but is not advised. On long cut and fill hills, drop inlets into storm sewers are sometimes necessary to prevent overloading of the ditch.

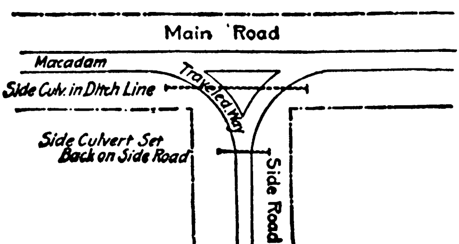


FIG. 47.—Case V. Side culverts.

Case V. Side Culverts (Fig. 47).—In designing culverts under side roads, the length must be great enough to provide an easy turn for traffic; many times a saving in length can be made by placing the culvert a short distance down the side road, as shown in Fig. 57, but this should, of course, not be done on steep grades.

Case VI. Bridge and Channel Locations (Figs. 48 and 49).—Figures 48 and 49 illustrate road approach relocation and simple

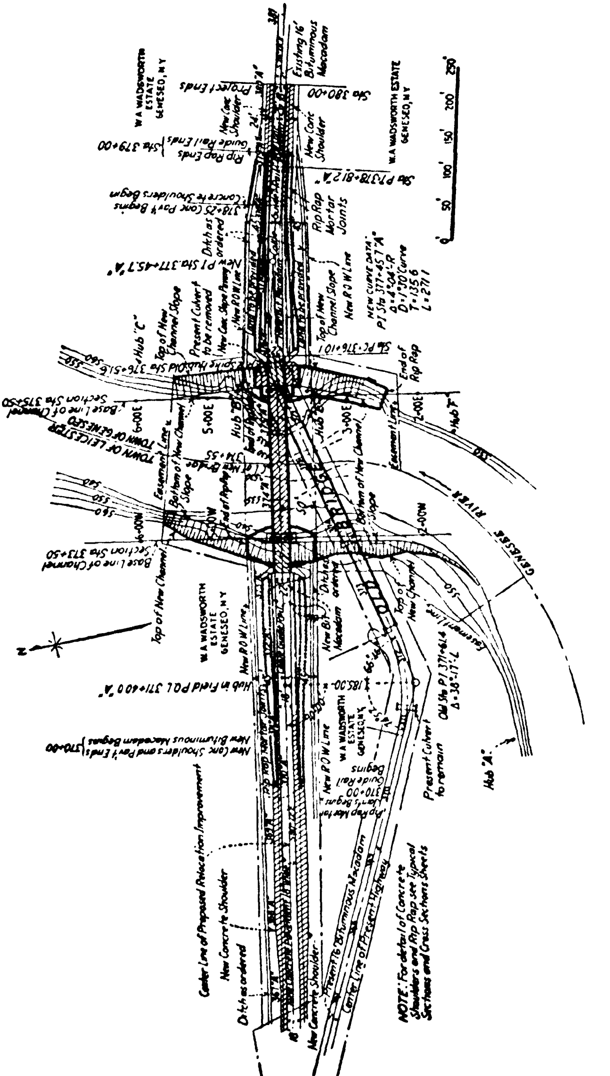


FIG. 48.—General layout bridge No. 5 road 1247. Showing improved road approach alignment, short right angle span and channel improvement.

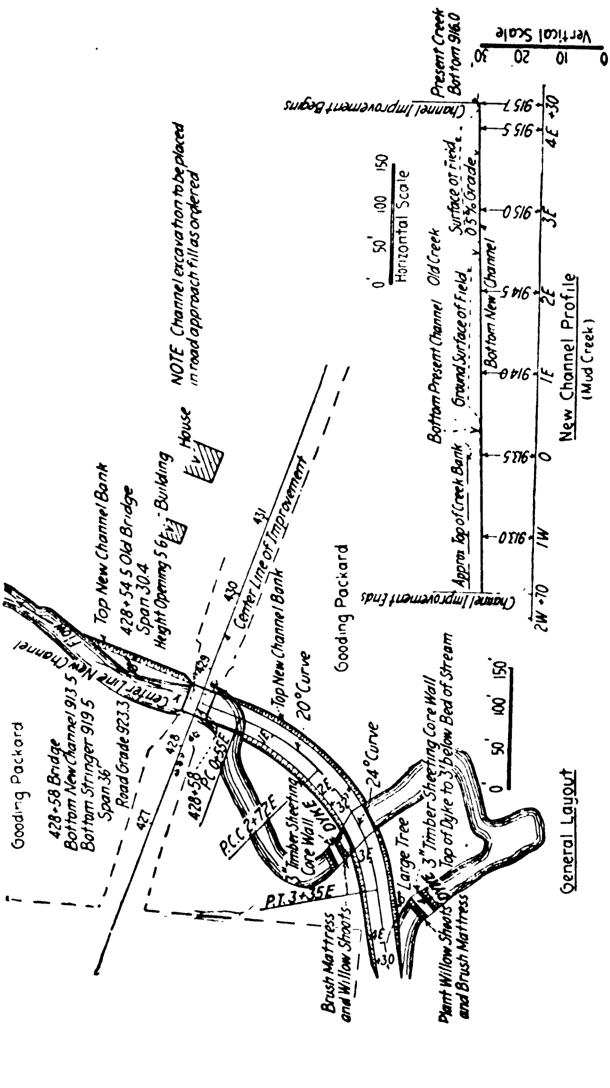


FIG. 49.—Channel improvement reducing required span and minimizing scour and silting at bridge site.

inexpensive channel improvement which betters road approaches, betters flow conditions, and reduces cost of structure.

Don'ts.—The main fault to be avoided in the location design of culverts and bridges is the use of the right-angle location where this is not the natural and reasonable layout. The right-angle layout is desirable on account of economy, where it fits the conditions.

Do not omit culverts on side-hill locations and run the water for long distances in the road ditches.

Waterway Area and Clearance for Ice and Débris.—The size of opening and elevation of bottom of superstructure (top of opening), depend on volume and velocity of stream flow and on clearance for ice and floating débris. They should generally conform to the natural conditions at the stream crossing and permit a normal flow at normal velocity.

Clearance for Ice and Débris.—For culverts and small-span concrete structures (less than 10′) no allowance is made for débris clearance, as it rarely does any damage to the structure, and short, small pondings during exceptional flow can be disregarded. On larger streams requiring spans of over 10′, a minimum clearance between bottom of structure and high-water surface is generally specified. Iowa specifies a minimum clearance of 1.0′ for spans of 12 to 40′ and 2′ for spans longer than 40′. Maine specifies a minimum of 4.0′ clearance for steel-girder or truss bridges. See page 1025 for Western New York débris clearance practice. A reasonable underclearance is a local matter and no broad rule can be given.

Volume and Flow of Water.—The size of opening is usually determined by noting the size of the old structure or, if none exists, the size of other structures over the same stream and by inquiries of neighboring residents or the road commissioner as to how the existing structure has handled the water in the past. As a general rule, the size of opening or span should not be reduced below that of the present structure, but in steel bridges that have been sold to town boards by enterprising bridge companies, it is often found that the span is needlessly long. The evidence of existing structures is the most reliable basis of design, but the conclusions should be checked theoretically, and for small drainage areas in villages and all drainage areas affecting new locations in sparsely settled districts either the physical evidence of high water or some maximum run-off formula must be used. Run-off formulas are based on the rate of rainfall, area of the watershed, topography, and soil. The rate of rainfall varies for different geographical locations and the length of the storm. Reliable information for any locality can be obtained from the Weather Bureau. Short storms develop the greatest intensity and produce the largest run-off for small watersheds. The rates reached by these storms should be considered in designing ditch relief culverts or cross-culverts with small drainage areas. A liberal basis for these cases is the 5- or 10-min. duration rate of Table 45 (p. 187). Table 46 (p. 188) illustrates the method. Most culvert design is based on a 24-hr. precipitation, as illustrated in Table 47 (p. 189), and applies to watersheds of, say, 0.5 sq. miles and up. Streams requiring structures of over 10′ span generally produce physical evidence of high water which can be safely used.

Table 50 (p. 192) gives the size of opening used by the Santa Fé R.R.; Table 48 (p. 190) gives the size of opening for small culverts used by the New York Central. Table 49 (p. 191) gives the size of culvert used by the Iowa State Highway Commission. These tables serve to illustrate the application of this principle of design.

Weather Bureau records show maximum 24-hr. precipitations of 7.66" at Portland, Ore.; 5.12" at Los Angeles, Cal., 2.06" at El Paso, Tex.; 7.03" at Kansas City, Mo.; 9.40" at New York City; and 8.57" at Savannah, Ga. These rates are rarely used for run-off computations as they represent extreme cases of rare occurrence. Good practice uses a 24-hr. rate of from 4 to 6". Openings based on these rates where the culvert will handle the water without quite running full will take care of unusual cases by the forced discharge due to the formation of a shallow pond on the upstream side of the road. Discharge velocity due to backwater head is shown approximately by Table 53 (p. 196).

TABLE NO. 45.—RATES OF RAINFALL. SHORT STORMS

Short storms of the greatest intensity occur as cloud-bursts in the mountain and arid regions between the Sierras and the foothills of the Rockies. The intensities of these storms are not well recorded but partial records indicate as high a fall as 11 inches in one hour. For these regions culverts for small drainage areas should be made at least twice as large as for eastern or southern conditions. (See last column, table No. 46.)

Maximum intensity of Rainfall for different periods taken from the U. S. Weather Bureau Records. Intensity at rate of inches per hour.

Location	5 Minute Duration	10 Minute Duration	One Hour Duration
Atlanta, Georgia	5.5 in.	5.5 in.	1.5 in.
Boston, Mass	6.7 in.	5.0 in.	1.7 in.
Chicago, Ill	6.6 in.	5.9 in.	1.6 in.
Cleveland, Ohio	5.6 in.	3.7 in.	1.1 in.
Denver, Colo	3.6 in.	3.3 in.	1.2 in.
Detroit, Mich	7.2 in.	6.0 in.	2.2 in.
Duluth, Minn	3.6 in.	2.4 in.	1.4 in.
Galveston, Tex	6.5 in.	5.6 in.	2.6 in.
Jacksonville, Fla	7.4 in.	7.1 in.	2.2 in.
Milwaukee, Wis	7.8 in.	4.2 in.	1.3 in.
Memphis, Tenn	6.6 in.	4.8 in.	1.9 in.
New Orleans, La	8.2 in.	4.9 in.	2.2 in.
Norfolk, Va	5.8 in.	5.5 in.	1.6 in.
Omaha, Neb	6.0 in.	4.8 in.	1.6 in.
Philadelphia, Penn	5.4 in.	4.0 in.	1.5 in.
Savanah, Geo	6.6 in.	6.0 in.	2.2 in.
St. Louis, Mo	4.8 in.	3.8 in.	2.3 in.
Washington, D. C	7.5 in.	5.1 in.	1.8 in.

Table 46.—Maximum Runoff. Small Watersheds Burkle-Ziegler, Sewer Formula

$$\text{Cubic feet per second per acre reaching culvert.} = C \times \left\{ \begin{matrix} \text{Av. cu. ft. rainfall} \\ \text{per second per acre} \\ \text{during heaviest fall.} \end{matrix} \right\} \times \sqrt[4]{\frac{\text{Av. slope of ground in feet per 1000}}{\text{No. of acres drained}}}$$

C = 0.75 for paved streets and built up business blocks.
C = 0.625 for ordinary city streets.
C = 0.30 for villages with lawns and macadam streets.
Assumed C = 0.25 for farming country. Note.—This value is high from the standpoint of sewer design but culverts are short and might better be liberal in size.

One inch of rainfall per hour equals 1 cu. ft. per second per acre.

Discharge in Cubic Feet per Second

Area in Acres	Rate of Rainfall 4″ per Hour						**Assumed Runoff Steep Stony Mountain Slopes
	Fall 5′ in 1000 *		Fall 20′ in 1000		Fall 50′ in 1000		Rainfall 8″ per Hour
	C = 0.30	C = 0.25	C = 0.30	C = 0.25	C = 0.30	C = 0.25	
1	1.8	1.5	2.5	2.1	3.1	2.7	6
2	3.0	2.5	4.2	3.5	5.4	4.5	12
3	4.1	3.4	5.7	4.8	7.2	6.0	18
4	5.0	4.2	7.2	6.0	9.0	7.5	23
5	6.0	5.0	8.5	7.1	10.7	8.9	28
6	6.8	5.7	9.7	8.1	12.2	10.2	33
7	7.7	6.4	10.9	9.1	13.7	11.4	38
8	8.5	7.1	12.0	10.0	15.1	12.6	42
9	9.3	7.8	13.2	11.0	16.5	13.8	46
10	10.1	8.4	14.3	11.9	18.0	15.0	50
20	16.9	14.1	24.0	20.0	30.2	25.2	90
30	23.0	19.2	32.5	27.1	40.7	33.9	120
40	28.5	23.8	40.3	33.6	50.9	42.4	150
50	33.6	28.0	47.7	39.8	60.0	50.0	180
60	38.6	32.2	54.6	45.5	68.7	57.3	200
70	43.3	36.1	61.4	51.2	77.3	64.4	225
80	48.0	40.0	67.9	56.6	85.2	71.0	250
90	52.4	43.7	73.9	61.6	93.1	77.6	275
100	56.7	47.3	80.2	66.8	100.8	84.0	300
200	95.4	79.5	134.6	112.2	169.7	141.4	550
300	129.0	107.7	182.9	152.4	229.7	191.4	750
400	160.0	133.6	227.0	189.2	285.6	238.0	880
500	190.0	158.0	268.0	223.5	336.6	280.5	980
600	216.0	180.0	307.0	256.0	387.0	322.8	1,050
640	230.0	*192.0	323.0	269.0	406.3	338.6	1,100

* 200 second feet by Table 47.
** Based on Santa Fe Table 50.

TABLE 47.—MAXIMUM RUNOFF, DICKENS FORMULA

$D = C\sqrt[4]{M^3}$ Runoff expressed in second feet. D = Discharge in cu. ft. per second. M = Drainage area in sq. miles. C = Coefficient.

The following tabulation is for a 24 hour precipitation of 4″ rain and for topography similar to the farming sections of the Eastern Atlantic States. For 6″ in 24 hours correct the quantities in proportion to C as follows.

4″ Rainfall		6″ Rainfall	
Flat Country	C = 200	Flat Country	C = 300
Rolling Country	C = 250	Rolling Country	C = 325
Hilly Country	C = 300	Hilly Country	C = 350

For steep stony watersheds and a 6″ rainfall use the Oklahoma Column of Table 50.

Area in Square Miles	Flat Country C 200	Rolling Country C 250	Hilly Country C 300
0.1 = 64 acres	36	45	54
0.2	60	75	90
0.3	81	101	121
0.4	100	125	150
0.5	119	149	180
0.6	136	170	204
0.7	153	191	229
0.8	169	211	253
0.9	185	231	277
1.0	200	250	300
2.0	334	417	501
3.0	456	570	684
4.0	564	705	846
5.0	668	835	1002
6.0	764	955	1146
7.0	860	1075	1290
8.0	950	1188	1426
9.0	1038	1297	1556
10.0	1122	1402	1682
20.0	1890	2362	2834
30.0	2560	3200	3840
40.0	3180	3975	4770
50.0	3760	4700	5640
60.0	4310	5400	6480
70.0	4840	6050	7260
80.0	5360	6700	8040
90.0	5840	7300	8760
100.0	6320	7900	9480

For areas under 0.1 square mile, see Table 46

NOTE.—For graph of this table see page 1021.

Examples of Use of Tables.—Table 51 (p. 194) gives the normal discharge of small culverts laid at different rates of grade. To illustrate the use of Tables 45 to 51, three examples will be given. Suppose water from 2 sq. miles of flat farming country in the North Atlantic states is to pass through a culvert having a natural slope of 0.5′ per 100′. Table 47 is figured for a 4″ rainfall in 24-hr., which is reasonable for this section. This table shows a run-off of 334 sec.-ft. for flat farm land. For a slope of 0.5′ per 100′, Table 51 shows that a 5 by 5′ culvert will carry the water.

Suppose steep, rocky ground is to be considered of, say, 200 acres or ⅓ sq. miles in Oklahoma and a culvert slope of 2′ per 100′. The best data are the Santa Fé (Table 50), which give an opening of 51 sq. ft. at 10′ per second or a run-off of 510 sec.-ft. Table 51 shows that a 5 by 4′ culvert on a 2% grade will carry this, but that the velocity is high and the culvert must have a solid bottom and rip-rap protection at both ends. Where pipes or solid bottom culverts are used, high velocity is not objectionable, but where the bridge type is used a sufficiently large opening to keep the velocity down to 10′ per second or less is advisable.

Suppose a ditch relief culvert drains 2 acres in the cloud-burst region and can be laid on a slope of 3′ per 100′. Use last column (Table 46), which gives 12 sec.-ft., which, from Table 51, gives a 16″ pipe.

Table 52 gives the approximate velocity of flow in ditches and streams knowing the cross-section area and slope of water surface.

Table 53 gives the approximate increase in elevation above a bridge in a stagnant-flow area to force water through the bridge opening at different velocities.

See also typical bridge reports (pp. 327 to 333) for determination of waterway area and débris clearance allowance.

TABLE 48.—NEW YORK CENTRAL & HUDSON RIVER R.R. CULVERTS FOR SMALL DRAINAGE AREAS

Steep, rocky ground, acres	Flat cultivation, long valley, acres	Size, diameter in inches	Equivalent capacity, pipes
5	10	10	
10	20	12	
20	40	16	
25	50	18	Two 16″ pipes
30	60	20	Two 16″ pipes
45	90	24	Two 18″ pipes
70	140	30	Two 24″ pipes
110	220	36	Two 30″ pipes
150	300	42	Two 30″ pipes
180	360	48	Two 36″ pipes
280	560	60	Two 36″ pipes

NOTE.—To be used only in the absence of more reliable information, particularly on existing culverts over the same stream.

TABLE 49.—CULVERT DESIGN, IOWA STATE HIGHWAY COMMISSION

Size of culvert opening, feet	Maximum acres	Minimum acres
2 by 2	70	28
4 by 4	376	140
6 by 6	1300	520
8 by 8	2700	1120
10 by 10	5000	2000

Iowa State Design Specifications, 1925 (Waterway Area)

1. **Determination of Size of Opening.**—In general, the determination of the size of the waterway opening required shall be made by the engineer from a study of the topographic and local conditions, available previous records of stream run-off, high-water, and other available data. The required field data for the determination of waterway opening are given in detail in a field manual entitled "Instructions for Making Bridge and Culvert Surveys." The general methods of determination of waterway openings shall be as follows:

a. Small Structures.—For small structures having waterway openings not greater than 400 sq. ft. the area of opening required may be determined by Talbot's formula, $A = C\sqrt[4]{D^3}$,
where A = area of waterway opening, in square feet.
C = coefficient:

Mountainous = 1.0
Hilly = 0.6 to 0.8
Rolling = 0.4 to 0.5
Flat = 0.2 to 0.3

D = drainage area, in acres.

b. Medium-sized Structures.—The size of opening required for medium-sized structures having areas of openings from 400 to 1000 sq. ft. shall be determined by the engineer from a study of the available data on topographic conditions, stream run-off, high water, and by a comparison of the values obtained from such studies with those obtained from other data, such as Talbot's formula, Dunn's table, etc. The use of an empirical formula alone in determining the size of opening required for structures of this size is not recommended.

c. Large Structures.—The waterway opening for structures having required openings greater than 1000 sq. ft. shall be determined by the engineer from a study of topographic conditions in the vicinity of the proposed structure, past records of high water and run-off, and the determination of probable stream run-off, using available data on the stream slope, mean velocity, and cross-sectional area of the stream below high water. A comparison should be made of the values thus obtained with the actual area of openings of existing structures on the same stream or on other streams under similar conditions. Empirical formulas having as factors only drainage areas and coefficients are not applicable in determining the size of openings for structures of this size.

TABLE 50.—SANTA FÉ RY. SIZE OF OPENING

Area Drained in Square Miles	Areas of Waterways Sq. Ft.			Area Drained in Square Miles	Areas of Waterways Sq. Ft.			Area Drained in Square Miles	Areas of Waterways Sq. Ft.			Area Drained in Square Miles	Areas of Waterways Sq. Ft.		
	Missouri and Kansas A	Okla-homa	Illinois		Missouri and Kansas A	Okla-homa	Illinois		Missouri and Kansas A	Okla-homa	Illinois		Missouri and Kansas A	Okla-homa	Illinois
0.01	2.0	2.0	East of Streator use 60% of Column A West of Streator use 80% of Column A	0.85	91.0	91.0	East of Streator use 60% of Column A West of Streator use 80% of Column A	4.2	403	409	East of Streator use 60% of Column A West of Streator use 80% of Column A	20	970	1062	East of Streator use 60% of Column A West of Streator use 80% of Column A
0.02	4.0	4.0		0.90	94.0	94.0		4.4	417	423		22	1015	1120	
0.03	6.0	6.0		0.95	97.0	97.0		4.6	430	436		24	1060	1170	
0.04	7.5	7.5		1.0	100.0	100.0		4.8	443	450		26	1100	1220	
0.05	9.0	9.0		1.1	110.0	111.0		5.0	455	464		28	1140	1275	
0.06	10.5	10.5		1.2	120.0	121.0		5.5	483	493		30	1180	1320	
0.07	12.0	12.0		1.3	130.0	131.0		6.0	509	519		32	1220	1370	
0.08	13.5	13.5		1.4	140.0	141.0		6.5	533	545		34	1255	1415	
0.09	15.0	15.0		1.5	150.0	151.0		7.0	556	572		36	1290	1460	
0.10	16.0	16.0		1.6	160.0	162.0		7.5	579	596		38	1320	1505	
0.15	25.0	25.0		1.7	170.0	172.0		8.0	601	622		40	1350	1575	
0.20	32.0	32.0		1.8	180.0	182.0		8.5	622	644		45	1435	1645	
0.25	38.0	38.0		1.9	190.0	192.0		9.0	641	666		50	1510	1745	
0.30	44.0	44.0		2.0	200.0	202.0		9.5	660	689		55	1580	1840	
0.35	51.0	51.0		2.2	220.0	223.0		10.0	671	709		60	1650	1930	
0.40	56.0	56.0		2.4	240.0	244.0		11.0	710	745		65	1720	2020	
0.45	62.0	62.0		2.6	260.0	264.0		12.0	740	781		70	1780	2100	
0.50	66.0	66.0		2.8	280.0	284.0		13.0	775	815		75	1840	2180	
0.55	70.0	70.0		3.0	300.0	304.0		14.0	805	857		80	1900	2255	
0.60	74.0	74.0		3.2	321.0	326.0		15.0	835	893		85	1980	2330	
0.65	78.0	78.0		3.4	340.0	345.0		16.0	865	930		90	2015	2405	
0.70	81.0	81.0		3.6	357.0	362.0		17.0	890	951		95	2065	2475	
0.75	85.0	85.0		3.8	373.0	379.0		18.0	920	998		100	2120	2545	
0.80	88.0	88.0		4.0	388.0	394.0		19.0	945	1020		110	2220	2690	

			East of Streator use 60 % of Column A West of Streator use 80 % of Column A				East of Streator use 60 % of Column A West of Streator use 80 % of Column A				East of Streator use 60 % of Column A West of Streator use 80 % of Column A				East of Streator use 60 % of Column A West of Streator use 80 % of Column A
120	2315	2820		300	3615	4690		850	5940	7640		2000	8820	11450	
130	2405	2955		325	3770	4860		900	6080	7875		2200	9240	11960	
140	2500	3080		350	3900	5040		950	6230	8170		2400	9605	12490	
150	2580	3240		375	4035	5220		1000	6380	8410		2600	9970	12995	
160	2665	3330		400	4165	5400		1100	6705	8720		2800	10370	13680	
170	2745	3445		450	4385	5660		1200	6960	9030		3000	10640	14370	
180	2820	3560		500	4610	5920		1300	7230	9340		3500	11445	15050	
190	2900	3665		550	4825	6175		1400	7480	9650		4000	12160	15740	
200	2970	3775		600	5030	6440		1500	7725	9960		4500	12825	16475	
220	3115	3980		650	5230	6695		1600	7960	10270		5000	13500	17275	
240	3245	4152		700	5420	6335		1700	8195	10580		5500	14080	18125	
260	3370	4330		750	5610	7170		1800	8390	10890		6000	14520	18975	
280	3495	4510		800	5800	7400		1900	8625	11200		6500	15140	19825	

NOTE.—The above classification by states is for convenience only, and merely denotes the general characteristics of topography and rainfall. Column A in table is prepared from observations of streams in southwest Missouri, eastern Kansas, western Arkansas and the southeastern portion of the Indian Territory. In all of this region, steep rocky slopes prevail and the soil absorbs but a small percentage of the rainfalls, and indicates larger waterways than are required in western Kansas and the level portions of Missouri, Colorado and New Mexico or Western Texas. This table is based on data procured from different western railroads, and from actual surveys and on a 6″ rainfall in 24 hours taken from Government statistics, with the understanding that most of it falls in 6 or 8 hours, and velocity under bridge or through opening is 10′ per second. From the above it is obvious that each waterway should be given a certain amount of individual consideration, and if it is desirable to depart from waterways in table, full explanation of conditions and reason for departure should be given.

Approved:

R. A. RUTLEDGE,
Chief Engineer.

See Vol. XI, No. 2, April, 1906, Journal Western Soc. of Engrs. for report on Dun's waterway table.
C. E. O., April 13, 1914.

TABLE 51.—APPROXIMATE DISCHARGE CAPACITY CAST-IRON PIPE AND SMALL CONCRETE BOX CULVERTS

Slope in Feet per 100	12" C. I. P. Vel.	12" C. I. P. Dis.	14" C. I. P. Vel.	14" C. I. P. Dis.	16" C. I. P. Vel.	16" C. I. P. Dis.	18" C. I. P. Vel.	18" C. I. P. Dis.	20" C. I. P. Vel.	20" C. I. P. Dis.	22" C. I. P. Vel.	22" C. I. P. Dis.	24" C. I. P. Vel.	24" C. I. P. Dis.
	Velocity in Feet per Second								Discharge in Cu. Ft. per Second					
0.5					4.5	6	5.0	9	5.4	12	5.8	15	6.2	19
1.0					6.8	9	7.0	12	7.7	17	8.2	22	9.2	29
2.0	7.4	6	8.3	9	9.2	13	10.0	13	10.9	24	11.5	30	12.3	39
3.0	9.2	7	10.3	11	11.2	16	12.2	21	13.4	29	14.2	37	14.8	46
4.0	10.6	8	11.8	12	13.1	18	14.2	25	15.3	34	16.5	43	17.2	54
5.0	11.8	9	13.3	14	14.6	20	16.2	28	17.3	33	18.0	43	19.0	60
6.0	13.0	10	14.6	15	16.2	23	17.5	31	19.0	41	20.0	53	21.3	67
Area Sq. Ft.	0.78		1.07		1.39		1.76		2.17		2.64		3.14	
Value of R	0.25		0.29		0.33		0.37		0.42		0.46		0.50	

CONCRETE BOXES

Slope in Feet per 100	2'×1.5' Vel.	2'×1.5' Dis.	2'×2' Vel.	2'×2' Dis.	3'×2' Vel.	3'×2' Dis.	3'×3' Vel.	3'×3' Dis.	4'×2' Vel.	4'×2' Dis.	4'×3' Vel.	4'×3' Dis.	4'×4' Vel.	4'×4' Dis.	5'×3' Vel.	5'×3' Dis.	5'×4' Vel.	5'×4' Dis.	5'×5' Vel.	5'×5' Dis.
0.5	7.0	21	7.7	31	8.6	52	9.7	87	9.7	78	10.8	130	11.6	186	11.9	179	13.0	260	13.8	340
1.0	10.0	30	10.4	42	12.5	75	13.7	123	13.7	110	15.0	180	16.7	267	16.9	254	18.5	370	19.0	475
2.0	14.0	42	14.8	59	17.5	105	19.0	171	19.0	152	22.0	264	23.6	378	23.8	357	26.2	523	28.0	700
3.0	17.2	52	18.3	73	21.6	130	23.5	212	23.5	188	26.5	318	29.0	464	29.5	443				
4.0	20.0	60	21.0	84	24.8	149	27.5	248	27.0	216	30.5	366								
5.0	22.6	63	23.5	94	27.5	165	31.0	279	31.0	248										
6.0	24.0	72	26.0	104	30.0	180														
Area Sq. Ft.	3.0		4.0		6.0		9.0		8.0		12.0		16.0		15.0		20.0		25.0	
Value of R	0.60		0.66		0.86		1.0		1.0		1.2		1.33		1.36		1.54		1.66	

Note: — Table 51 is figured from Church's diagrams of Kutters formula using n = 0.011; the use of these diagrams for short culverts is approximate only but it is sufficiently close for the purposes for which this table is intended.

TABLE 52.—APPROXIMATE MEAN VELOCITIES OF FLOW

(In feet per second)

Coefficient of friction N = 0.03 channel in fair condition.

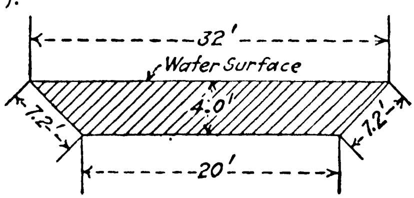
$$R = \text{hydraulic radius} = \frac{\text{Water flow area (square feet)}}{\text{Wetted perimeter of channel (Linear feet)}}.$$

(Compiled from Church's diagrams of Kutter's formula)

Slope of channel in feet per 1000′	Hydralic radius R										
	0.2	0.4	0.6	1	2	3	4	6	8	10	20
0.01	....	....	....	.....	0.22	0.32	0.42	0.6	0.8	0.92	1.7
0.1	....	0.18	0.28	0.42	0.75	1.0	1.3	1.7	2.2	2.4	4.0
0.2	0.16	0.28	0.42	0.61	1.08	1.45	1.75	2.4	2.9	3.4	5.4
0.3	0.19	0.36	0.51	0.77	1.3	1.75	2.2	2.9	3.5	4.1	6.4
0.4	0.22	0.42	0.59	0.9	1.5	2.05	2.45	3.4	4.0	4.7	7.3
0.5	0.26	0.48	0.66	1.0	1.7	2.25	2.8	3.75	4.5	5.3	8.2
1.0	0.37	0.68	0.94	1.4	2.4	3.2	3.95	5.2	6.3	7.3	11.5
2.0	0.52	0.96	1.35	2.0	3.4	4.6	5.6	7.3	9.1	10.4	16.5
3.0	0.64	1.2	1.7	2.5	4.2	5.6	6.9	9.0	11.0	13.0	19.5
4.0	0.74	1.3	1.85	2.8	4.8	6.4	7.9	10.2	12.8	14.5	22.5
5.0	0.83	1.5	2.15	3.2	5.4	7.2	8.9	11.6	14.0	16.5	25.5
6.0	0.91	1.7	2.3	3.5	5.9	7.9	9.8	12.8	15.8	17.7	27.5
7.0	0.98	1.8	2.5	3.8	6.4	8.6	10.5	13.6	17.0	19.0	29.5
8.0	1.04	1.9	2.7	4.1	6.9	9.2	11.2	14.5	18.0	20.5	32.0
9.0	1.10	2.05	2.8	4.3	7.3	9.7	11.9	15.5	19.0	22.0	34.0
10.0	1.19	2.2	3.0	4.5	7.7	10.2	12.6	16.5	20.0	26.0	36.0
15.0	1.4	2.7	3.7	5.5	9.5	13.0	15.5	20.0	25.0	28.0	
20.0	1.7	3.1	4.3	6.4	10.9	14.5	18.0	23.0	28.0	33.0	
30.0	2.1	3.8	5.2	7.9	14.5	18.0	22.0	28.0	35.0		
40.0	2.4	4.3	6.1	9.1	15.5	21.0	25.0	33.0			
50.0	2.7	4.8	6.8	10.1	17.5	23.0	28.0	37.0			
100.0	3.8	6.8	9.5	14.2	24.0	33.0	40.0				

NOTE.—For coefficient of friction of 0.035, channels in poor condition, use 85 % of the velocities given in the above table.

Example of Use of Table 52.—To find velocity of flow and carrying capacity of improved channel having a 20′ bottom width, 1 on 1½ side slopes, and a 4′ depth of water. Channel on grade of 0.1% (1′ per 1000′).



$$\text{Flow area} = \frac{32 + 20}{2} \times 4 = 104 \text{ sq. ft.}$$

$$\text{Wetted perimeter} = 20 + 7.2 + 7.2 = 34.4'$$

$$\text{Hydraulic radius } R = \frac{104}{34.4} = 3.0.$$

Channel in fair condition $N = 0.03$

From table, mean velocity of flow for $N = 0.03$, $R = 3.0$, and fall of 1.0′ per 1000′ equals = 3.2′ per second.

Carrying capacity of channel = 104 sq. ft. × 3.2 = 332.8 cu. ft. per second.

For channel in poor condition use velocity for $N = 0.035$, which is 85% of 3.2 = 2.7, 104 × 2.7 = 281 cu. ft. per second.

TABLE 53.—APPROXIMATE BACKWATER HEAD IN FEET, TO GIVE DIFFERENT VELOCITIES OF FLOW THROUGH BRIDGE OPENING

$$H = \frac{V^2}{2g} = \frac{V^2}{64.4} \text{ or } V = \sqrt{64.4\,H}$$

Approximate backwater head, feet	Forced velocity, feet per second
0.1	2.5
0.2	3.6
0.3	4.4
0.4	5.1
0.5	5.7
0.6	6.2
0.7	6.7
0.8	7.2
0.9	7.6
1.0	8.1
1.5	9.8
2.0	11.4

Practical Considerations Governing the Size of Waterway.—For moderate-sized drainage areas the culvert opening is proportioned to the run-off, but for small areas the size is determined by the convenience of cleaning rather than by the discharge capacity. Where sufficient fall can be obtained to make the culvert self-cleaning, a 12″ pipe is feasible under shallow fills, but where the flow is sluggish, nothing less than a 16 to 18″ pipe will serve satisfactorily. Long culverts under deep fills should never be smaller than 2′ wide and 3′ high to permit cleaning by hand.

The self-cleaning velocity of flow for sand and earth particles is about 1′ per second; for coarse gravel, about 3′ per second (Ogden's "Sewer Design," p. 134). A pipe laid on a slope that gives a velocity of 5′ per second when flowing one-quarter full should keep clean. This requires a fall of approximately 2′ per 100′ for a 12″ pipe and is the minimum grade at which the 12″ size should be used as a short culvert. Long storm sewers use lighter grades (see Table 61, p. 338).

It is probable that a culvert should have the same slope as the stream bed. If given a greater slope, the outlet end tends to clog, and if a lesser, the inlet end will plug. It is unusual for culverts to fill badly except when placed at the foot of a steep hillside where the stream velocity is materially reduced. At such points an extra large structure should be designed with the idea of providing sufficient waterway even after the contraction caused by this settlement has occurred. Such a culvert should be cleaned after each freshet. The use of short, paved dips in the roadway at such points in place of culverts is not advised, as they are dangerous and cause accidents unless very gradual. A man not familiar with the road often loses control of his car.

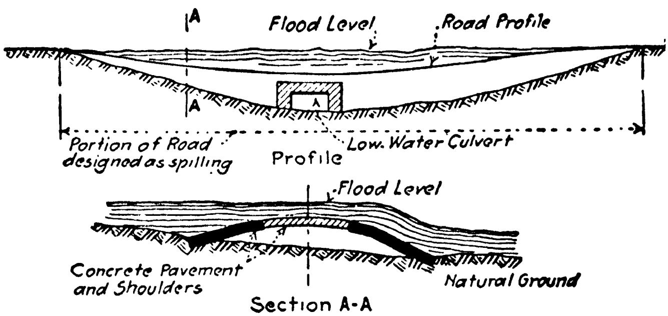


FIG. 50.—Overflow and low water culvert.

If, however, too much trouble is experienced in carrying large infrequent floods under the road, a small culvert can be used for the low-water flow which does not, as a rule, carry much silt, and the flood flow can be carried over the roadbed by paving the entire surface with concrete from toe of slope to toe of slope and giving the longitudinal road profile a slight dip safe for traffic to localize the flooded portion of the road (Fig. 50).

More trouble is experienced from culverts becoming filled with ice due to alternate freezing and thawing weather. This is particularly true of small culverts draining springs. Culverts as large as 2 by 2′ have frozen solid in this manner, and if this difficulty is anticipated the size should be regulated accordingly or trouble will be experienced during the spring break-up. The following ingenious expedient has been successfully used on roads where the culverts fill with ice and snow during the winter. A small pipe is suspended inside of the normal culvert. In the fall this small pipe is plugged, and in the spring just as the snow begins to melt the plugs are removed and the first water flowing through the small pipe melts the ice and snow rapidly for the entire length of the culvert so that it is generally completely free to handle the main spring run-off.

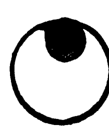


FIG. 51.—Small pipe in culvert.

Grade and Elevation of Culvert Inverts.—As previously stated, it is desirable to prevent silting up of the culvert due to abrupt changes in the velocity of flow. For this reason culverts are normally given the same slope as the stream bed.

The elevation of the invert is always made low enough to drain all surface water from the upstream adjacent lands and, if the elevation of the outlet permits, it is desirable to make the culvert low enough to act as an outlet for farm underdrains. In hilly country, underdrainage need not be given much weight, but in flat country it often controls the elevation of the culvert invert.

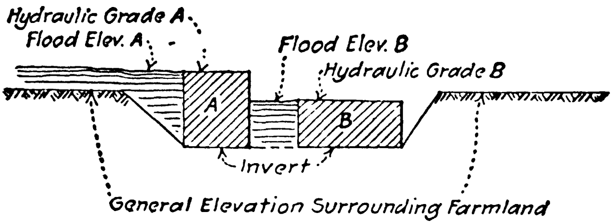


FIG. 52.

In order to prevent serious ponding and damage to crops in flat country, all culverts or bridges on channels of any importance should be placed at such an elevation that the top of the waterway opening is as low or lower than the surrounding farm land; that is, the culvert elevation and shape of opening should be designed for the hydraulic grade of maximum flood flow.

Figure 52 illustrates this point. The waterway areas of two culverts *A* and *B* are the same in size. In order to get the full capacity

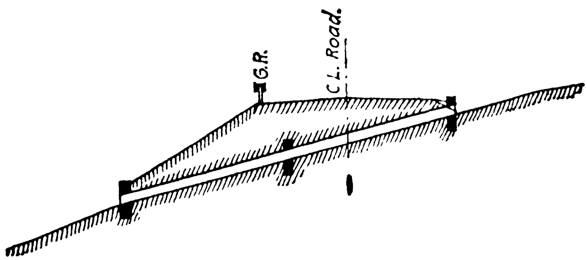


FIG. 53.

of *A*, however, the water would have to back up and overflow the surrounding lands. Culvert *B* carries the flood flow without serious ponding.

The use of bridge openings similar to *A* is very common practice in both railroad and highway design, as it generally cheapens the culvert, but is undesirable, causing needless damage to the abutting properties.

Where pipe culverts are laid on steep slopes, special buttresses, well imbedded in the natural hard slope, should be provided to prevent crawl or slip. Well-built headwalls should hold up to, say, 12° side slope, and beyond this limit extra anchors should be provided (see Fig. 53).

Depth of Cushion under Pavement.—A cushion of earth between the top of a concrete culvert or pipe and the bottom of the pavement is desirable. This is more important where the pavement is a rigid type, such as brick or concrete, than where it is a macadam construction.

The depth of this cushion sometimes controls the culvert invert elevation where the topography and road grade make a low invert needlessly expensive or impracticable.

For rigid pavements the minimum desirable depth of cushion is approximately 6″. This cushion between culvert top and bot-

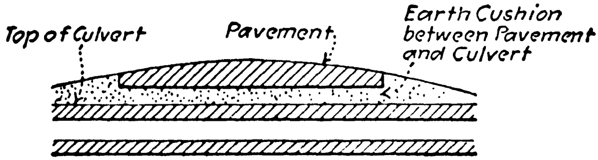


FIG. 54.—Cushion over culvert slabs.

tom of pavement should be gravel or macadam to prevent frost heave due to saturated earth. If less than this is used the chances are that the pavement will crack over the side walls of the culvert unless the pavement base is specially thickened and reinforced with steel. For macadam pavements, a 6″ cushion is desirable, but no great damage occurs if the bottom course is laid directly on the culvert top. Even with a cushion, settlement often develops each side of culverts having less than 2 to 3′ of cover, but it can be easily fixed by the maintenance gang.

Dead and Live Loads. *Dead Loads.*—Dead loads include weight of structure plus earth and pavement fills over the superstructure. They are readily determined. Weights of material making up dead loads are given in Table 172 (p. 1038). Dead-load computations should include future pavement surfacing to repair bridge floors. If the type of flooring used can be removed and replaced without additional thickness and weight, no extra allowance is required; if a solid concrete floor is used which in the future will be recapped with additional depth of asphaltic concrete or block pavement, the future extra weight must be considered.

Earth-fill loads on superstructures of culverts or bridges are usually computed for total depth between vertical planes at the ends of the structure and make no allowance for arch action of earth fills. Some designers reduce this load for arch action on the basis of inclined planes at an angle of about 20° with the vertical at the ends of the structure; but there have been enough cases of trouble with culverts under deep fills to warrant conservative assumptions in regard to vertical earth pressures, and the total earth weight between vertical planes seems justified as a basis of design.

Live Loads.—Live loads include weight of traffic plus impact allowance and are largely a matter of judgment. There is quite a range in the values used. It is universally conceded, however, that at least four classes of loading should be considered to meet variations, in local conditions prevailing in the United States and that it is neither wise nor economical to apply heavy-traffic loading to light-traffic bridges, as is quite frequently done by broad arbitrary standardized rulings of state highway departments.

Specifications issued by the U. S. Bureau of Public Roads prepared in conjunction with the Association of State Highway Officials are given below, as they probably represent the trend of present practice as closely as any data which could be given. Equivalent uniform loads for different spans are given in the chapter on Office Design (p. 1040).

"**Highway Loads for Bridge Floors.** (*American Association of Highway Officials*).—The integral parts of bridge-floor systems, including their direct connections to trusses, girders, and viaduct towers and bents, shall be designed for the maximum stresses produced by truck concentrations. Floor beams, hangers, and all integral members or parts of trusses and girders

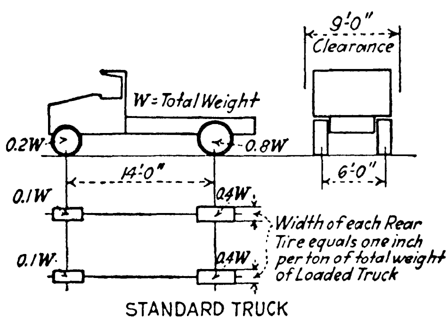


FIG. 55*A*.—Truck dimensions.

subjected to the direct action of floor loads and impacts shall correspondingly be designed for truck concentrations. The truck dimensions and weight distributions used for design purposes shall be those of the typical or standard trucks shown in Fig. 55. These standard trucks are designated by the letter H, followed by a numeral indicating for each class its gross or total weight in tons.

"The class of loading used shall be one of the following:

Loading H-20	20-ton trucks
Loading H-15	15-ton trucks
Loading H-12½	12½-ton trucks or an alternate load of one 15-ton truck
Loading H-10	10-ton trucks

"**Highway Loads for Truss and Girders.**—The trusses and girders of bridge spans and the columns of viaduct towers and bents shall be designed for the stresses produced by a load on each traffic lane composed of a uniform load per linear foot of lane with a concentrated load so located longitudinally therein as to produce maximum stresses. The concentrated load shall be considered as uniformly distributed transversely on a line having a length equal to the width of the lane. The standard truck clearance width of 9′ shall be assumed as constituting the width of one traffic lane.

"The class of loading used shall be one of the following:

"*Loading H-20.*—A total load on each traffic lane composed of a uniform load of 600 lb. per linear foot and a single concentrated load of 28,000 lb.

"*Loading H-15.*—A total load on each traffic lane composed of a uniform load of 450 lb. per linear foot and a single concentrated load of 21,000 lb.

"*Loading H-12½.*—A total load on each traffic lane composed of a uniform load of 375 lb. per linear foot and a single concentrated load of 17,500 lb.

"*Loading H-10.*—A total load on each traffic lane composed of a uniform load of 300 lb. per linear foot and a single concentrated load of 14,000 lb.

"NOTE.—Loading H-15 is 75%, loading H-12½ is 62½%, and loading H-10 is 50%, of loading H-20.

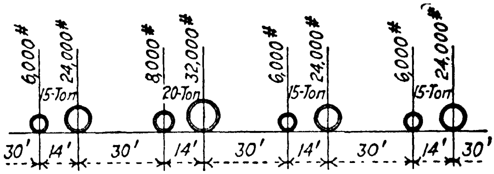


FIG. 55*B*.—Spacing of trucks.

"Loading H-20 is approximately equivalent to the typical truck loading shown in Fig. 55*B*. This loading consists of one 20-ton truck followed by, or preceded by, or both followed and preceded by, a line of 15-ton trucks of indefinite length and assumed to occupy a clearance or lane width of 9'.

"**Electric Railway Loads.**—When highway bridges support electric railway traffic in addition to highway traffic, the railway loading shall be selected with due regard to the class of traffic which may be expected to operate over the railway lines. Special consideration shall be given to the possibility that the freight rolling stock of steam railroads may be operated.

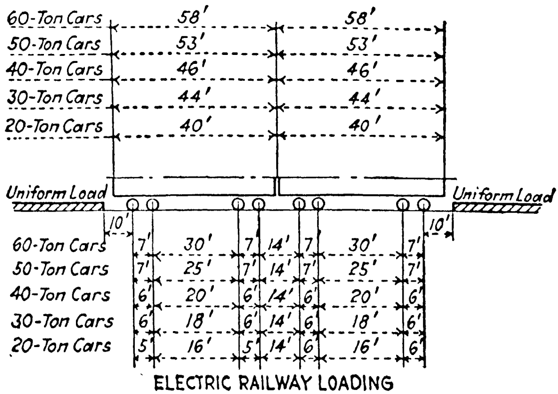


FIG. 55*C*.—Car loadings (electric).

"When not otherwise specified, the electric railway loading on each track shall be a train of two electric cars followed by, or preceded by, or both followed and preceded by, a uniform load. The cars shall be of one of the classes shown in Fig. 55*C*, the designation indicating the total loaded weight of each car, with corresponding axle spacings as indicated. The uniform load per foot of track shall be the weight of the loaded car assumed in the design divided by twice the overall car length. The portion of the roadway width assumed to be occupied by the railway loading shall have a width of 10'.

"For freight car loading, the typical cars shown in Fig. 55*D* may be assumed in the absence of more exact data.

"The railway loading used shall be shown on the stress sheets.

"Highway bridges supporting electric railway traffic shall be designed for the following loading conditions:

"1. The highway loads upon any portion of the roadway area, including that portion occupied by the railway.

"2. The electric railway loads on the car tracks and the highway loads on the remaining traffic lanes.

"**Load Classification of Bridges.**—Bridges shall be classified or rated in relation to their capacities for safely supporting highway loads or a combination of highway and electric railway loads. In general, the division into classes and the corresponding loadings shall be as follows:

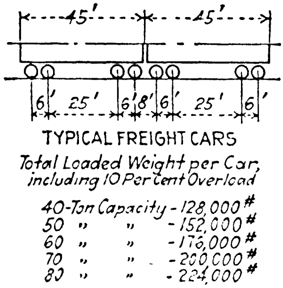


FIG. 55*D*.—Typical freight cars.

"*Class AA.*—Bridges supporting specially heavy traffic units in locations where the passage of such loads is frequent. Class *AA* bridges shall be designed for loading H-20.

"*Class A.*—Bridges supporting normally heavy highway traffic units with occasionally heavy loads. Class *A* bridges shall be designed for loading H-15.

"*Class B.*—Bridges supporting normally light highway traffic units with occasional heavier loads. Class *B* bridges shall be designed for loading H-12½.

"*Class C.*—Bridges of a temporary or semitemporary nature supporting light highway traffic untis. Class *C* bridges shall be designed for loading H-10.

"*Class D.*—Bridges supporting electric railway traffic in addition to highway traffic. The highway loads may correspond to any one of the four classes above specified. The electric railway loads shall be as specified."

These loadings are safe for military purposes, as the following statement of Major General W. M. Black, Chief of Engineers, 1917, will show.

"Our existing ordnance liable to accompany a field army will have its heaviest representative in a 12" howitzer weighing about 27,000 lb., 18,600 lb. of which are on the front wheels. The base or distance between the front and rear axles is 18'; width of track, 7' 4"; width of tire, 8"; width of tire shoes, 12". The howitzer is drawn by a 75-hp. caterpillar tractor weighing 25,000 lb. Comparison with the largest present-day commercial trucks shows that a road or bridge substantial enough for such will suffice for the ordnance load."

Impact.—The extra design load allowance for impact of moving loads is subject to considerable difference in practice, ranging from 0 to 60% of the static weight of the vehicles. Specifications of common practice are given on pages 1040 and 1549.

As a general rule, the following values represent conservative practice in impact allowance.

Floor systems of bridges having timber plank floor surface, 30 to 50%.

Floor systems of bridges and trestles having concrete floor slabs supported by stringers or floor beams, 20 to 30%.

Impact allowance for girders and trusses (see p. 1040).

Impact allowance for floor beam connections (see p. 1040).

For culverts and filled spandrel arch bridges, where the earth cushion between the top of the bridge slab or the arch and bottom of the road pavement exceeds 1′, the impact allowance disappears (0% impact).

Distribution of Loads.—Rules for current practice in load distribution on concrete slabs, stringers, and floor beams are given on pages 1042 to 1043.

Permissible Stresses in Materials.—This phase of design is quite well established and the common values are given in the chapter on Office Design (p. 1033).

The following table gives safe loads directly for simple structural elements:

Steel I-beams	(pp. 1062 to 1067).
Timber beams	(pp. 1076 to 1077).
Concrete beams	(pp. 1072 to 1074).
Concrete slabs	(pp. 1072 to 1073).
Timber columns	(p. 1078).
Effect of depth of fill on culvert slabs	(pp. 224 and 1075).

Roadway Widths and Clearances.—It is desirable to use the normal roadway width at culverts and bridges so that there is no extra hazard at these points. This general principle can usually be applied without excessive expenditure for culverts and for short-span bridges up to about 25′ span. Beyond 40′ span it is customary to reduce bridge roadway width to the minimum width, which can be safely used by traffic without reduction in speed or danger of side swipe as long-span bridges are a costly element of construction; between 20′ and 40′ spans, roadway widths between minimum and normal are used to meet conditions. Separate sidewalks, at least 5′ wide, are provided for pedestrians in well-settled districts, and are well segregated from road traffic by a raised curb and rail or by placing them outside of trusses or girders. Minimum vertical portal clearance is usually set at 13′ 6″ to 14′ for trucks and 16.0′ for electric cars.

Single-track or one-way bridges are rarely justified except for pioneer roads or local farm roads carrying a very small amount of traffic. Double-track (two-lane) bridges are required on the score of safe passing for practically all roads carrying from 100 to 5000 vehicles daily (10 hour count in summer). Above 5000 vehicles daily a three-lane bridge will serve satisfactorily up to 7000 to 9000 daily (10 hour count in summer) and beyond this limit a four-lane bridge is required. City street bridges should use the normal street width. *In designing bridge roadway widths an allowance of 60 years growth in traffic volume is reasonable for types of bridges which cannot be easily widened. For easily widened types a 20 to 30 year period is ample. (See page 32 for methods of estimating future traffic volume.)*

For two-lane traffic the A.S.C.E. recommends 20′ minimum roadway. The Hoover Committee on National Safety recommends 22′ minimum where motor busses use the road. For additional traffic lanes, 9′ is added for each additional lane.

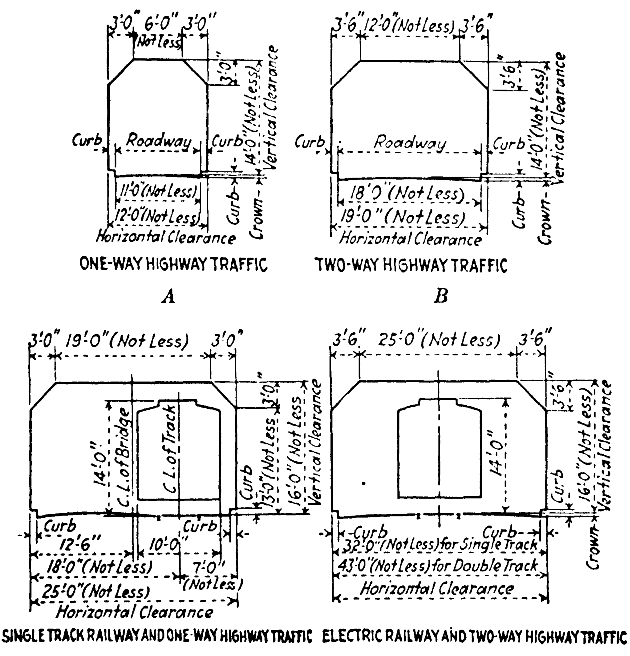
Table 54 gives minimum bridge widths recommended by various authorities for long-span bridges (over 40′ span) on state road systems.

Table 54.—Minimum Roadway Widths
(Bridge spans over 40′)

	One-lane, feet	Two-lane, feet	Extra for additional traffic lanes, feet
U.S. Bureau of Roads	12	18	9
A.S.C.E	12	20	9
Hoover Committee of Safety	. .	22	9
State of Iowa[a]	16	18–20	9
State of New York	.	22	10
State of Illinois	.	22	9

[a] For structure less than 40′ span minimum, two-lane traffic roadway width 24′.

The following clearance diagram (Fig. 56) is from the U. S. Bureau of Public Roads Specifications.



Figs. 56*A–D*.—Bridge clearance diagrams. U. S. Bureau Public Roads Standards (1926).

The following quotation from Illinois 1925 practice shows its sliding scale of bridge widths for different classes of roads.

ROADWAYS ON BRIDGES, ILLINOIS PRACTICE, 1925

All grade separations.............................Minimum 24'
Bridges and culverts:
I. On state highways
  1. 30' or less between abutments on center line..............................Minimum 30'
  2. More than 30' between abutments on center line..........................Minimum 22'
II. On state-aid roads
  1. 12' or less between abutments on center line..............................Minimum 30'
  2. More than 12' between abutments on center line..........................Minimum 20'
III. On principally traveled roads
  1. 12' or less between abutments on center line..............................Minimum 24'
  2. More than 12' between abutments on center line..........................Minimum 18'
IV. On secondary roads
  1. 12' or less between abutments on center line..............................Minimum 20'
  2. More than 12' between abutments on center line..........................Minimum 16'

NOTE.—For spans of more than 12' between abutments, hand rails not less than 3' above the crown of of roadway shall be used.

Type of Structure.—Reasonable type selection depends on ease of construction, appearance, future widening, depth of floor system, foundations, proper use of piers, an economical combination of abutments and different types of superstructures and the utilization of old structures.

Ease of Construction.—Ease and simplicity of construction are desirable, particularly for local road structures under control of the ordinary town superintendent of highways, where expert engineering supervision and competent inspection are not available. For these conditions cast-iron pipe for culverts, rolled-steel, I-beam bridge superstructures with simple concrete slab flooring and mass concrete abutments and wings are preferable types, as they can be abused in construction without much loss of strength. This limitation does not apply to work designed and supervised by expert engineering forces.

Appearance.—Appearance of structure does not often control design on rural highways, but under some conditions, such as parks, cemeteries, village locations, it must be considered. Concrete arches or other types of concrete slab bridges with ornamental balustrades look much better than steel-girder or open-truss bridges.

Future Widening.—Future widening has considerable influence on the type of bridge superstructure and layout of abutments and wings. As traffic volume often grows rapidly beyond expected limits, and as local funds are often too small to make it possible to build the bridge wide enough for all future requirements, it is often wise to use a type of bridge that can be widened in the future without entirely rebuilding. Slabs, stringers, arches, and deck girders can be easily widened. All types of through girders and trusses cannot be widened without complete rebuilding. Abutments and wings with their faces in a straight line with step-down, vertical plane offsets can be easily extended without loss of previous work.

Depth of Floor System.—In many cases it is undesirable and costly to raise the approach grades needlessly and for such conditions a type of bridge with the least depth of floor system is desirable; that is, type design considers floor depth as it affects cost and suitability of approaches as well as the bridge structure itself. It is a common fault for the bridge designer to think only of the economic design of the structure and to neglect the cost of approaches This is particularly true of railroad-grade-crossing-elimination structures. Where it is desirable to keep the floor depth to a minimum the most economical designs are usually through girders with floor slab supported directly on the floor beams spaced sufficiently close so that the concentrated slab load is carried by at least three floor beams. The following table gives the approximate depth of floor systems for different types of bridges of different roadway widths (H-20 loading) and different spans.

APPROXIMATE DEPTH OF FLOOR SYSTEMS (INCHES)
(Road grade to bottom of superstructure)

Type of superstructure, H-20 loading	Span and roadway widths							
	Spans							
	10′		25′		45′		100′	
	Roadways							
	20′	30′	20′	30′	20′	30′	20′	30′
Concrete slabs	16″	16″	28″	28″				
Steel I-beam stringers	26″	26″	38″	38″	42″	42″		
Concrete T-beam stringers	....	....	31″	31″	62″	62″		
Reinforced-concrete arches	18″	18″	21″	21″	23″	23″	32″	32′
Through plate girder, floor slab, beams and stringer	....	....	....	....	40″	50″	40″	50″
Through plate girders, floor slab and floor beams[a]	....	....	....	....	34″	44″	34″	44″
Through plate girder, floor slab on extra short spaced floor beams[b]	....	....	....	....	28″	38″	28″	38″

[a] This is generally the most economical type of floor system for girder bridges for usual approach conditions (see Fig. 77, p. 290).

[b] This type of floor system is generally economical for long, high approaches.

Foundations. Soils.—Rock or hardpan foundations generally indicate rigid frame slab types or arches as an economical solution, provided there is an adequate height for a reasonable rise ratio. Arches of over 10′ span require piles for any foundation except rock or hardpan and are not usually economical nor conservative design where piles are required, as slight settlement is often disastrous to the arch type and adequate pile foundations are very costly. Arches of less than 10′ span are often designed with bottom ties (sewer section) and used on ordinary soil foundations, but such design (false arches) seems to be an effort to use this type for conditions for which it is not primarily suited.

Ordinary foundation soils (clay, sands, gravels, etc.) generally make it desirable to select a type of superstructure which transmits its load vertically to the abutments and which will not be materially damaged by slight settlement. These types are the slab, stringer, or girder bridges with free end supports, and even for these types it is desirable to use piles for spans of over 40′ for any soil except gravel hardpan or rock or for shorter spans where much scour is likely to occur.

Safe foundation loads on soils according to different authorities are given in Table 55. For important structures the actual bearing value of the soil should be determined as given in the chapter on Inspection (p. 1337).

Piles.—The kind of pile and its safe loading depend on physical ground-water conditions and the type of structure to be supported.

In general, it is inadvisable to use untreated timber piles unless they are below the permanent ground-water level. Treated timber is necessary to resist sea water, borers, and for trestles or where the pile is alternately wet and dry. Concrete piles are desirable where piles are not located below permanent ground-water level.

Piles supporting arches or continuous concrete beams or girders which are injured by even small settlement must be driven to practical refusal and given a lighter individual load than where they support structures with free ends which are not injured by small settlement.

Table 56 gives current-practice safe design loads for different kinds of piles for different kinds of structures. The actual safe load on piles is determined by test loads or by empirical formulas as described in detail in the chapter on Inspection (p. 1338).

Minimum penetrations, spacing and detail dimensions, and construction manipulation are given in Specifications (p. 1338).

Abutments and Piers.—The use of piers is generally confined to low-height stream crossings where the stream flow is sluggish and ice jams are infrequent. The minimum spacings of piers recommended by the 1925 Iowa Specifications are as follows:

TABLE 55.—SAFE LOAD ON FOUNDATION SOILS IN TONS (2000 LB.) PER SQUARE FOOT OF BEARING AREA

Authority	Quicksand and alluvial silt	Clean, dry sand	Confined sand and fine gravel	Coarse gravel	Ordinary mixture sand and clay	Soft, damp clay	Dry clay	Cemented gravel or hardpan	Soft rock	Hard rock
Baker's "Foundations"......	½	2	4	....	.....	1	4	8	5	25
Hool and Johnston..........	½–1	2–4	4	....	.....	1–2	4–8	8–10	5–10	15–30
Ketchum........	.....	3	4	4–5	2	....	3–4	....	8	20
State of Iowa....	.....	....	3	3	1.5	....	3	5	7	25
State of Washington........	0–½[a]	3	3–3½	4–7	1½–2	0–2	3–5	5–8	8–15	25
New York Building Code......	.....	3	4	4	2	1	4			
Division 4, New York State Highway	½	2	3	4–5	2	1	3–4	6–7	8	20

[a] Confined.

TABLE 56.—DESIGN LOAD ON PILES (TONS PER PILE)

Type of structure	Type of pile	Ordinary load	Extreme maximum load
Slab, girder or trusses...	Timber	12–15	20
Slab, girder or trusses......	Concrete	25	30
Trestles and viaducts........	Timber	15	20
Trestles and viaducts..............	Concrete	25	30
Arches or continuous girders[a].......	Timber	10–12	15
Arches or continuous girders[a].......	Concrete	20	25

[a] Piles for these structures to be driven to practical refusal (see p. 1339, Part 2), 25- to 30-ton resistance.

NOTE.—For minimum dimensions of piles see Specifications (p. 1501). Pile spacing generally 3 to 3½′ C. to C. Maximum 4′ without special reinforcing design. Spacing closer than 2½′ worthless. Increase size of footer to get minimum spacing of 2½′.

IOWA SPECIFICATIONS, 1925

"2. **Channel Opening.**—In general, the channel opening provided shall be equal to or greater than that required to discharge the maximum flow safely without damage to the structure from scour.

"*a. For Streams Having Deep Channels.*—In general, the channel opening for structures over streams having deep channels and well-defined banks not subject to overflow shall be of such length that it will entirely span the main channel. In streams in which the banks are subject to possible future widening, provision shall be made for such conditions by an increase in span length.

"*b. For Streams Having Low Banks and Subject to Overflow.*—On streams having low banks which are subject to overflow and where the approach to the structure is above high water, the waterway opening provided shall be sufficient to discharge the flood area at safe velocities against scour and the main channel openings shall be of sufficient length to discharge débris and ice efficiently.

. . .

"4. **Pier Spacing.**—On streams having a difference in height between high and low water exceeding 6', the main channel piers of multiple-span bridges shall not be spaced closer to each other than four times the difference of height between high and low water. The above shall not apply to a stage which is caused by backwater."

Where piers are permissible, they generally result in the use of a series of slab or stringer superstructures in place of single-span girders or trusses. They also permit multiple arches which are pleasing in appearance.

For spans of less than 45 feet piers must be used with caution but where it is certain they will not cause clogging they often result in a material reduction in cost due to less floor depth and cheaper superstructures. They are particularly desirable for low height openings in rock or hardpan foundations. This same general statement applies to the use of double box culverts with central pier wall where a concrete floor is desirable to prevent scour on high velocity small streams.

Mass concrete is common practice for short-span (less than 50') bridge abutments and piers, and is preferable for even high long-span structures, although reinforced-concrete buttress abutments are often used for high abutments. Abutment footings are usually carried down at least 4' below the stream bed in ordinary soils, and pier footing at least 6'. Where hard rock is encountered, the footing is usually 6" to 1' below the surface of the rock in order to key in properly, and where soft rock or hardpan is encountered the bottom of the footing is usually at least 2' below the surface of the hard strata. Figures 69 to 78 show common practice for mass abutments and piers for bridges up to 100' spans.

Superstructures.—Table 57 shows the general limits of span for which different types of bridge may be considered in making

TABLE 57.—LIMITATIONS OF USE OF DIFFERENT TYPES OF CULVERTS AND BRIDGES

Type	Range in span
Pipes	12–42"
Box culverts	2–10'
Reinforced-concrete slabs	5–25'
Steel I-beam stringers	10–55'
Reinforced-concrete stringers (T-beams)[a]	20–40'
Reinforced-concrete through girders	20–50'
Reinforced-concrete arches (hardpan or rock foundations)	All spans above 6'
Steel-plate girders (deck or through)[b]	40–100'
Steel pony trusses	40–100'
Steel-riveted through truss	Above 90'
Steel pin connected through truss	Above 150'
Timber structures:	
Trestle spans	10–25'
Truss spans	Above 25'

[a] T-beam reinforced-concrete stringers not economical where forming is difficult due to soft foundations or high abutments.

[b] Deck girders rarely advisable due to clearance limitations. Where they are permissible, they are generally cheaper than the through type.

alternate estimates of cost. These alternate estimates of costs must include initial cost plus maintenance and renewal costs, considering a reasonable length of life for the different types. Maintenance painting for exposed steel for 1925 cost conditions amounts to about 50 to 70 cts. per ton per year average, which, capitalized at 5%, is equivalent to approximately $10 to $15 per ton which must be added to the initial cost of construction to determine final comparative costs. For steel weights see bridge steel diagrams, Chap. IX (p. 652). Renewal costs are very indefinite, as the comparative life of steel and concrete structures cannot be closely approximated. If steel structures are well maintained, their lives should compare favorably with concrete or steel encased in concrete. As a matter of fact, the maintenance is often neglected, which materially shortens the life of steel bridges.

Utilization of Old Structures.—The proper utilization of old structures, either in whole or in part, has a decided effect on materially reducing the cost of bridge replacements. There is too much tendency towards complete abandonment and entire new construction. To illustrate the value of careful utilization of well-built parts of old structures we quote the following paragraph from the report of the Bridge Engineer of N. Y. State Highway Dept. Div. 4 for 1926.

"Thirty-five small span bridges (less than 45′ span) have been designed in 1926. In thirteen cases it was possible to utilize some part of the old structure and such action has reduced construction cost of these bridges approximately $30,000."

Examples of common methods of utilizing old structures are given in Chap. XIV, pages 1013 to 1020.

Consideration of utilizing the old existing structure completes the factors which determine the selection of economic and suitable type of bridge to be used. The typical design report on page 327 illustrates the application of the principles discussed for bridge type selection. The quick cost estimating diagrams, pages 310 to 326, are helpful in making comparative cost estimates of bridges. Desirable culvert types and relative costs are given on page 213.

Surface of Bridge Floors.—On rural highways, bridge floors are usually constructed of reinforced concrete, the upper 3″ of which is considered as the pavement surface and is not considered as adding anything to the strength of the slab. This upper 3″ is, however, generally molded monolithic with the slab proper and is reinforced with standard paving mesh. A recent modification of this standard procedure constructs the standard bridge floor slab first, waterproofs the top, and then constructs a thin pancake concrete surface (see Fig. 76). The older practice of monolithic molding is much superior, and when a new surface is needed some form of asphaltic concrete may be used on thin blocks on the top of the pitted concrete floor. Any thin layer of concrete will crack up readily under heavy loading unless it is at least 5″ to 6″ thick well reinforced and there is no advantage in eventually renewing this pancake and replacing at the same elevation, as the road pavements on both sides of the bridge will in the future be capped and raised in the same manner as recommended for floor repair: the original design should provide for a small increase in future dead load to provide or the future floor capping (25 lb. per square foot). Thin asphalt block or bituminous concrete makes the most satisfactory renewable surface (see Specifications, p. 1563).

Bridge Parapets.—For city and special bridges extremely ornate balustrades are used but for rural highways it is desirable to use the simple types shown in Figs. 69 to 76. Of these types the level-top, paneled parapet is generally the most satisfactory as it fits in well with the straight-line approach guard railing and it is comparatively easy to finish properly which is important considering the class of masons generally employed on road work. The type shown in figure 76*A* is all right provided there is no approach straight-line guide rail but it does not look well in conjunction with approach rail.

Parapets along sidewalks should be from 38 to 40″ above the walk to insure ample protection.

Condemnation and Repair of Weak Bridges.—For data on this phase of bridge work the reader is referred to Chap. XII, page 779.

Protection from Scour.—Where the natural stream velocity is high or where it is necessary to restrict the bridge opening with a resultant high velocity which causes bank scour, rip-rap or other protection must be provided. Specifications for rip-rap are given on page 1506.

Iowa 1925 Specifications give the following maximum velocities requiring special bank protection for different natural soils:

"**Restricted Waterways.**—If the area of waterway provided is not sufficient to discharge the maximum flood flow at mean velocities less than the following, ample provision shall be made in the design for protection against scour:

a. Sand, alluvial soil, and soft clay.................. 4′ per second
b. Firm clay, loose stones, or boulders.............. 6′ per second
c. Rock, shale, or cemented clays.................. 8′ per second."

Rip-rap protection reduces scour. According to Trautwine, a velocity of 8 m.p.h. or 12′ per second will not derange quarry rubber stones exceeding ½ cu ft. deposited around piers or abutments. Specifications for rip-rap are given on page 1506. If the natural stream velocity is not over 10′ per second the span is usually regulated so that the velocity under the bridge during freshets will not exceed 10′ per second. If the natural stream velocity of flow at the bridge site is not known, it can be approximated roughly for small streams by the formula from Table 52 (p. 195).

Paved Fords.—For wide, shallow arroyos of the arid regions of the West, paved fords are in general use. These channels carry water only during sudden severe storms, and to provide structures large enough to carry the sudden large infrequent flows would be practically prohibitive in cost. The road across an arroyo is kept slightly below the natural elevation of the wash and is paved with concrete, cobblestone, or timber (see sketch). The alignment is straight and the location of the pavement is shown during flood by four marking posts, two at each end, which also indicate the depth of water, so that it can be used even if covered with water, unless the depth is too great for safety, which can be determined by the gages on the range posts. As the concrete is below the bottom of the stream, no scour occurs and generally a thin layer of sand is deposited on the concrete, which can be easily cleaned off with a road machine.

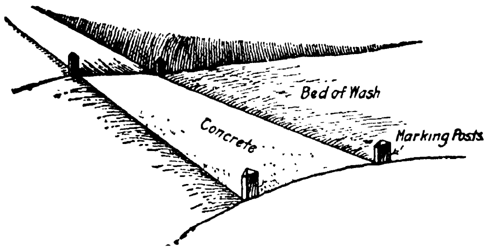


FIG. 57.—Paved ford.

Example of Current Practice in Design.

Culverts (pp. 214 to 236).

Small-span bridges (pp. 237 to 309).

Pipe Culverts.—The pipe culverts in common use are as follows (see Figs. 59 to 63, pp. 215 to 219) for typical pipe):

Corrugated metal..................	semipermanent construction
Vitrified tile........................	semipermanent construction
Reinforced-concrete pipe............	semipermanent construction
Vitrified tile incased in concrete.....	permanent construction
Reinforced-concrete pipe cradled in concrete........................	permanent construction
Cast-iron pipe.....................	permanent construction

These types of culverts are suitable on firm foundations and are generally economical for small drainage areas. The relative cost will fluctuate for each contract, which makes it impossible to generalize as to the economy of selection.

Box Culverts.—The two general types favored are the plain masonry or concrete bottom and side wall with reinforced cover slab, and the lighter box reinforced on all sides (see Figs. 65 and 67).

Circular-opening culverts are sometimes used (see Fig. 68).

Relative Economy of Culverts.—Comparative estimates of cost must be made for each contract, but to give a general idea of the method of economical selection Table 58 is inserted for 1926 cost conditions prevailing in western New York.

The semipermanent types should not be used on high-class improvements except for driveway culverts.

Comparative estimates similar to Table 58 furnish a rational basis for judgment, provided only the permanent types are compared and that the comparison is made for each contract considering the special conditions prevailing due to location, market quotations on materials, and local materials available.

For conditions similar to Table 58, western New York, 1926, permanent pipe culverts are not economical over an 18″ diameter. For drainage areas requiring a culverts waterway area of over 2 sq. ft. the box type is preferable. Of the box types the simple mass concrete structures are more satisfactory on firm foundation soils, considering construction difficulties, than the thin-wall rein forced type, although they cost somewhat more than the thin side-wall type. This, however, is a matter of personal judgment.

The thin-wall type, reinforced in bottom, sides, and top, is preferable to the mass type under very deep fills or on soft foundations where bottom slab action occurs or where negative corner bending moments must be considered. (See figure page 224).

TABLE 58.—APPROXIMATE RELATIVE COST OF PIPE AND BOX CULVERTS 30′ LONG INCLUDING HEADWALLS (EXCLUSIVE OF EXCAVATION)

Size culvert pipe, inches	Area waterway, square feet	Style of construction					
		Corrugated metal	Vitrified tile	Vitrified tile incased in concrete	Reinforced-concrete pipe	Reinforced-concrete pipe cradled in concrete	Cast-iron pipe
12	0.78	$ 60	$ 50	$ 65	$ 75	$ 90	$100
14	1.07	70	60	75	80	95	130
18	1.76	90	80	110	100	130	170
24	3.14	120	120	160	135	175	260
30	4.88	140	180	230	180	230	
36	7.05	170	260	310	240	290	
42	9.60	200	...	...	300	360	
48	12.52	230					

Size culvert opening span-height, feet	Area waterway, square feet	Style of construction, concrete boxes			
		Mass concrete bottom, and sides (Fig. 65)		Thin reinforced sides and bottom (Fig. 66)	
		Total cost	Cost per square foot waterway	Total cost	Cost per square foot waterway
2 by 1.5	3	$160	$57		
2 by 2	4	190	50	$150	$35
3 by 2	6	210	37		
3 by 3	9	260	30	200	22
3 by 4	12	300	26		
4 by 2	8	250	31	230	30
4 by 3	12	300	25		
4 by 4	16	340	21	300	20
5 by 3	15	350	23		
5 by 4	20	400	20	350	17
5 by 5	25	450	18		

The following cuts illustrate typical practice in small-culvert and short span bridge design.

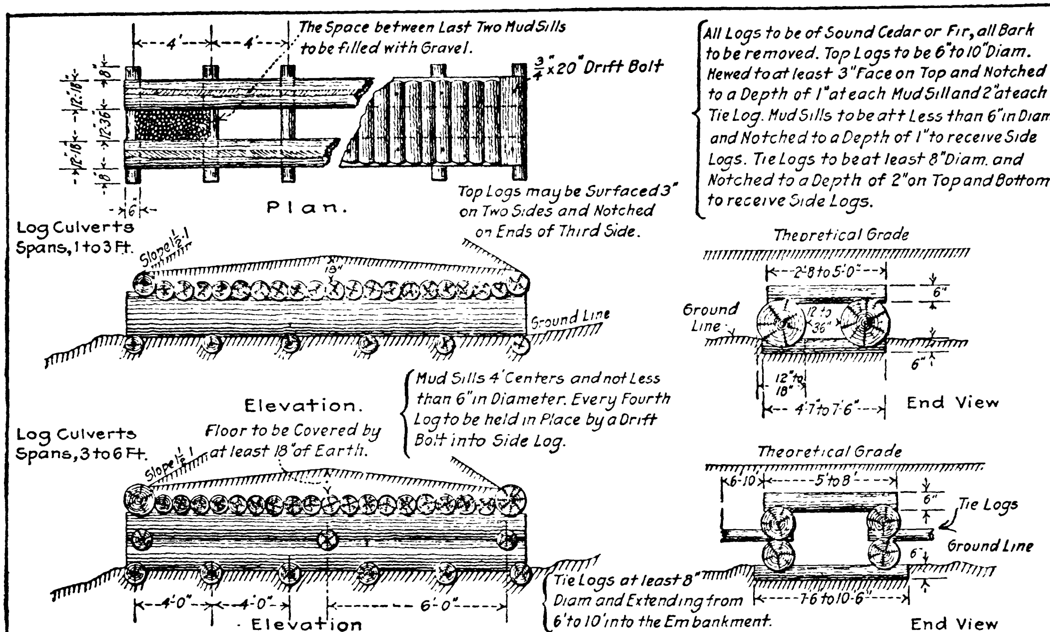
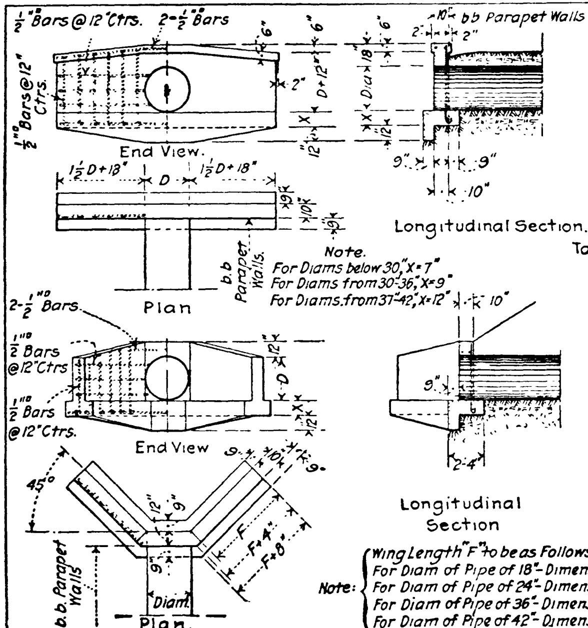


Fig. 58.—Typical log culverts pioneer roads.



Note: Wing Length "F" to be as Follows:—
For Diam of Pipe of 18"—Dimension F to be 4'-0"
For Diam of Pipe of 24"—Dimension F to be 4'-6"
For Diam of Pipe of 36"—Dimension F to be 5'-0"
For Diam of Pipe of 42"—Dimension F to be 5'-6"

Table of Specifications for Sheet Metal Culverts.

Diameter	Min. Sheet Thickness	Min. Rivet	Max Rivet Spacing	Remarks
12 in	1/4 in	1/2 in	6 in	Farm Crossings only
15 "	1/4 "	1/2 "	6 "	
18 "	1/4 "	1/2 "	6 "	
24 "	1/4 "	1/2 "	6 "	
30 "	1/4 "	5/8 "	6 "	
36 "	3/8 "	5/8 "	6 "	
42 "	7/16 "	5/8 "	6 "	

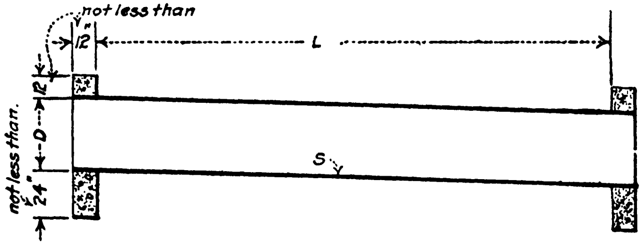
Table of Specifications for Corrugated Culverts.

Diameter	Min Gage	Min Rivet	Max Rivet Spacing	Remarks.
12 In	US Std. Pl and Iron and Steel Gage 16 lb	3/8 in	One Rivet in each Corrugation	This Size for Farm Driveways
15 "	14 "	3/8 "		
18 "	14 "	3/8 "		Not to be used where Fill over Culvert exceeds 6 ft
24 "	14 "	3/8 "		
30 "	12 "	3/8 "		Not to be used where Fill over Culvert exceeds 4 ft
36 "	12 "	3/8 "		
42 "	12 "	3/8 "	2 in ea. Cor	

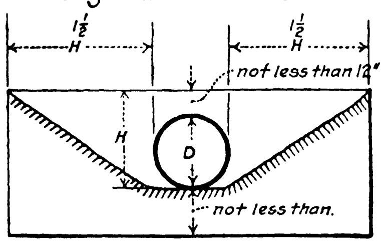
16 lb., 14 lb. Etc.

Note: Corrugated Pipe for use on Township Roads and Farm Driveways only, or for Temporary Repairs on County Roads.

FIG. 59.—Metal pipe culverts. State of Iowa.



Longitudinal Section



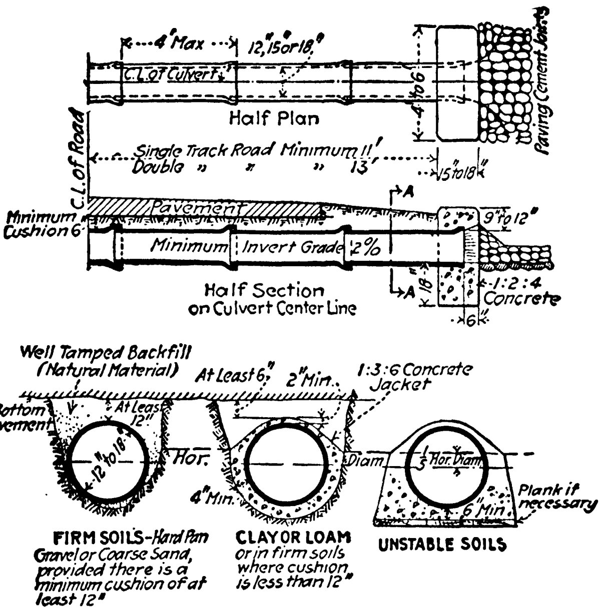
End Elevation.

TABLE OF PROPERTIES.

Diameter "D"	Slope "S"	Capacity, Cu Ft. per Sec.	Concrete Cu. Yds.
10"	0.048	1.64	1.75
12"	0 033	2 36	2.0
14"	0.025	3.21	2 3
16"	0.020	4.20	2 8
18"	0 016	5.31	2.9
20"	0.012	6 54	3 2
24"	0.010	9.42	3.8
30"	0.007	14.73	4.9
36"	0 005	21.21	6.1
Velocity = 3.0 Ft. per Sec. "n" = 0.027			

Quantities Figured from Minimum Dimensions.

FIG. 60.—Corrugated metal culverts. State of New Hampshire.



SECTION A-A-Showing Treatment in Different Soils

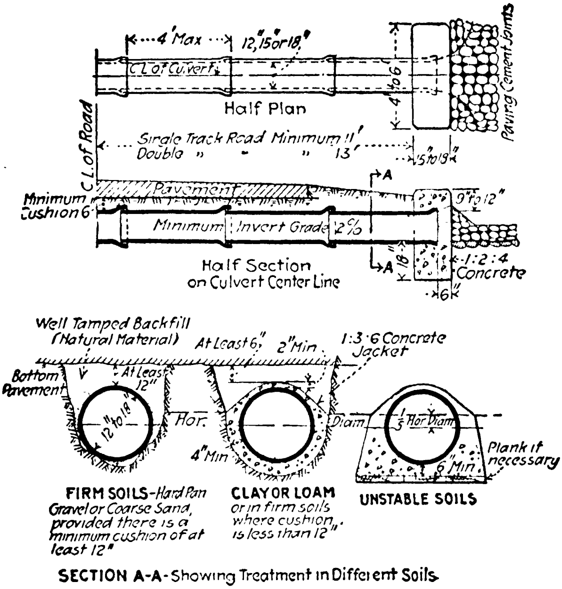
APPROXIMATE WEIGHT, DIMENSIONS, ETC. OF STANDARD SEWER PIPE

Calibre, in.	Price per foot	Weight per foot, lbs.	Depth of socket, in.	Annular space, in.	Thickness, in.
12	$1.35	45	2 1/4	1/2	1
15	1.80	60	2 1/2	1/2	1 1/8
18	2.50	85	2 3/4	1/2	1 1/4
20	3.00	100	3	1/2	1 3/8
22	4.00	130	3	1/2	158
24	4.50	140	3 1/4	1/2	158

DOUBLE STRENGTH PIPES

Calibre, in.	Price per foot	Weight per foot, lbs.	Depth of socket, in.	Annular space, in.	Thickness, in.
15	$1.80	75	2 1/2	1/2	1 1/4
18	2.50	118	2 3/4	1/2	1 1/2
20	3.00	138	3	1/2	1 3/8
22	4.00	157	3	1/2	1 5/6
24	4.50	190	3 1/4	1/2	2

FIG. 61.—Typical vitrified pipe culverts.



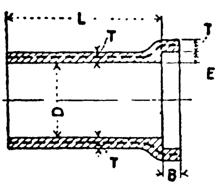


TABLE OF DIMENSIONS AND REINFORCEMENT FOR PIPE

Table of Dimensions

D Inches	L-Max. Feet	T Inches	B-Min. Inches	E Inches
12	4	2	2½	2½
15	4	2	2½	2½
18	4	2½	3	3
24	4	3	3	3½
Effective Area of Circumferal Reinforcement Per Foot Length of Pipe				
12	0.058 Sq. Inches			
15	0.058 " "			
18	0.080 " "			
24	0.126 " "			
Approximate Weight Per Linear Foot of Pipe				
12	90 lbs			
15	110 "			
18	170 "			
24	260 "			

FIG. 62.—Typical reinforced concrete pipe culverts.

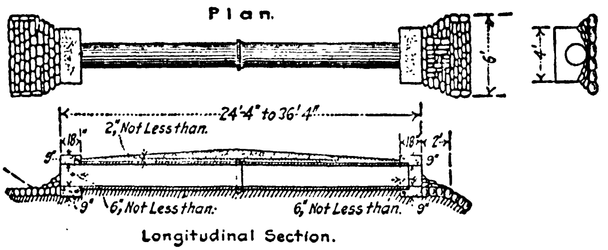
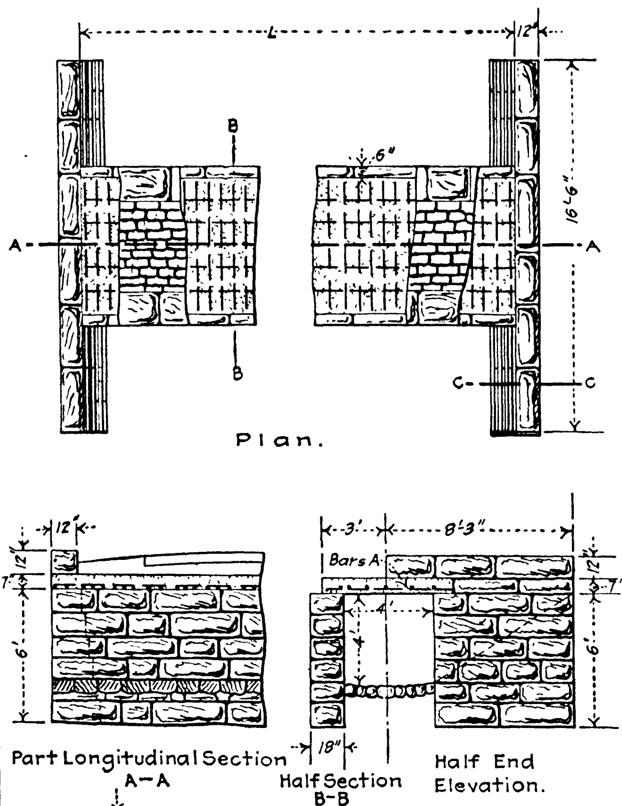
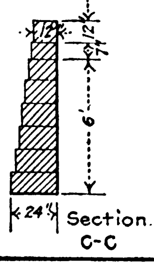


FIG. 63.—Cast iron pipe culvert. New York State standard.

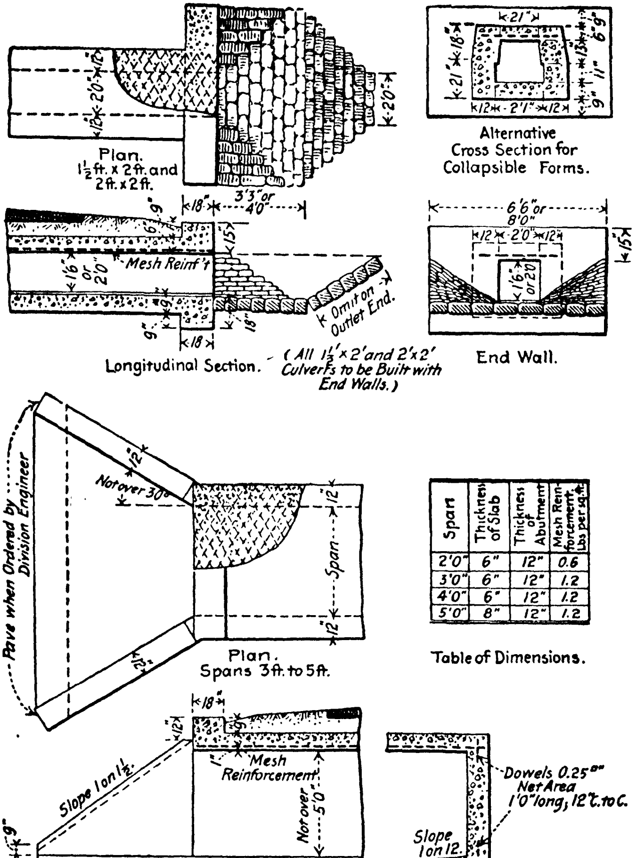




Depth Fill	Length L.	Bars A ½" Sq. Spacing	Number	Length	Masonry Cu. Yds.	Concrete Cu. Yds	Paving
1	21'-0"	7"	40	5'-9"	24.3	3.0	10.2
2	21'-6"	6½"	43	"	24.6	3.0	10.4
3	24'-6"	7½"	43	"	26.6	3.4	11.8
4	27'-6"	7"	51	"	28.6	3.8	13.1
5	30'-6"	7"	56	"	30.6	4.2	14.4
6	33'-6"	6½"	66	"	32.6	4.6	15.8
8	39'-6"	6"	83	"	36.6	5.4	18.4

TABLE OF QUANTITIES 4'x4' CULVERT

FIG. 64.—Typical masonry culvert. State of New Hampshire.

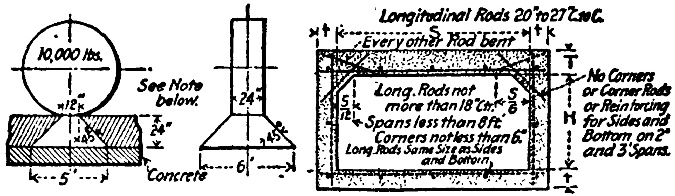


Span	Thickness of Slab	Thickness of Abutment	Mesh Reinforcement Lbs per sq. ft.
2'0"	6"	12"	0.6
3'0"	6"	12"	1.2
4'0"	6"	12"	1.2
5'0"	8"	12"	1.2

Table of Dimensions.

FIG. 65.—New York State small box culverts. Suitable for firm foundation soils. On soft soils use reinforced culverts, Fig. 66, page 222 and Fig. 67, page 224.

Note.—For effect of deep fills on culvert design see page 224. These culverts safe for following maximum depth of fill over top of slab assuming firm foundation soil. 2' span no limit, 3' span 30 ft. limit, 4' span, 20 ft., 5' span 13 ft. limit.



Assumption for Live Load. Cross Section of Culvert.

FIG. 66.

TABLE FOR STANDARD CULVERTS

Loading 15-ton road roller, 10,000 lbs. on each wheel. Unit stresses 16,000 lbs. for steel, 600 lbs. for concrete, 0.7% of steel. Los Angeles Co. Highway Com., A. E. Loder, Chief Eng.

				Top reinforcement			Corner reinforcement			Side walls reinforcement			Bottom reinforcement			Quan. per lineal ft. box		
D[1]	H[2]	T[3]	t[4]	Size	Spc.	Lgth.	Size	Spc.	Lgth.	Size	Spc.	Lgth.	Size	Spc.	Lgth.	C. yds. con.	Steel lb.	
8"	1'	4"	4"	3/8"	8"	2'- 6"										.091	3.70	2-span
8"	2'	4"	4"	3/8"	8"	2'- 6"										.115	3.70	
8"	1'	5½"	5"	3/8"	5"	3'- 9'										.155	7.17	
8"	2'	5½"	5"	3/8"	5"	3'- 9"										.186	7.17	3'-span
8"	3'	5½"	5"	3/8"	5"	3'- 9"										.216	8.12	
8"	1'	6"	5"	1/2"	8"	4'- 8"	3/8"	16"	1'-6"	3/8"	16"	1'-9"	3/8"	16"	4'- 8"	.204	12.83	
8"	2'	6"	5"	1/2"	8"	4'- 8"	3/8"	16"	1'-6"	3/8"	16"	2'-9"	3/8"	16"	4'- 8"	.235	13.55	4'-span
8"	3'	6"	5"	1/2"	8"	4'- 8"	3/8"	16"	1'-6"	3/8"	16"	3'-9"	3/8"	16"	4'- 8"	.266	15.22	
8"	2'	6½"	5"	1/2"	7"	5'- 8"	3/8"	14"	1'-6"	3/8"	14"	2'-9"	3/8"	14"	5'- 8"	.278	17.91	
8"	3'	6½"	5"	1/2"	7"	5'- 8"	3/8"	14"	1'-6"	3/8"	14"	3'-9"	3/8"	14"	5'- 8"	.309	19.68	5'-span
8"	4'	6½"	5"	1/2"	7"	5'- 8"	3/8"	14"	1'-6"	3/8"	14"	4'-9"	3/8"	14"	5'- 8"	.339	20.49	

8"	2'	7½"	6"	½"	6"	6-10"	3/8"	12"	1'-9"	3/8"	12"	3-0"	3/8"	12"	6'-10"	.373	23.23	
8"	3'	7½"	6"	½"	6"	6-10"	3/8"	12"	1'-9"	3/8"	12"	4'-0"	3/8"	12"	6'-10"	.410	25.14	6'-span
8"	4'	7½"	6"	½"	6"	6'-10"	3/8"	12"	1'-9"	3/8"	12"	5'-0"	3/8"	12"	6'-10"	.448	26.10	
8"	2'	8½"	6"	5/8"	8"	8'-10"	½"	16"	2'-0"	½"	16"	3'-2"	½"	16"	8'-10"	.494	38.32	
8"	3'	8½"	6"	5/8"	8"	8'-10"	½"	16"	2'-0"	½"	16"	4'-2"	½"	16"	8'-10"	.531	41.30	8'-span
8"	4'	8½"	7"	5/8"	8"	9'-0"	½"	16"	2'-0"	½"	16"	5'-2"	½"	16"	9'-0"	.628	43.02	
8"	5'	8½"	7"	5/8"	8"	9'-0"	½"	16"	2'-0"	½"	16"	6'-2"	½"	16"	9-0"	.671	46 00	
8"	2'	10"	7"	5/8"	6½"	11'-0"	½"	13"	2'-6"	½"	13"	3'-3"	½"	13"	11'-0"	.700	54 90	
8"	3'	10"	7"	5/8"	6½"	11'-0"	½"	13"	2'-6"	½"	13"	4'-3"	½"	13"	11'-0"	.742	58 17	
8"	4'	10"	7"	5/8"	6½"	11'-0"	½"	13"	2'-6"	½"	13"	5-3"	½"	13"	11'-0"	.786	59 74	
8"	5'	10"	8"	5/8"	6½"	11'-2"	½"	13"	2'-8"	½"	13"	6'-4"	½"	13"	11'-2"	.900	63.90	10-span
8"	6'	10"	8"	5/8"	6½"	11'-2"	½"	13"	2'-8"	½"	13"	7'-4'	½"	13"	11'-2"	.949	65 47	
8"	7'	10"	8"	5/8"	6½"	11'-2"	½"	13"	2'-8"	½"	13"	8'-4"	½"	13"	11'-2"	1.000	68 74	
8"	8'	10"	8"	5/8"	6½"	11'-2"	½'	13"	2'-8"	½"	13"	9'-4"	½"	13"	11'-2"	1.048	70.31	
8"	2'	11"	8"	5/8"	6"	13'-2"	½"	12"	3'-0"	½"	12"	3'-7"	½"	12"	13'-2"	.916	69.31	
8"	3'	11"	8"	5/8"	6"	13'-2"	½"	12"	3'-0"	½"	12"	4'-7"	½"	12"	13'-2"	.965	72 71	
8"	4'	11"	8"	5/8"	6"	13'-2"	½"	12"	3'-0"	½"	12"	5'-7"	½"	12"	13'-2"	1.014	74.41	
8"	5'	11"	8"	5/8"	6"	13'-2"	½"	12"	3'-0"	½"	12"	6'-7"	½"	12"	13'-2"	1.063	77.81	
8"	6'	11"	9"	5/8"	6"	13'-4"	½"	12"	3'-0"	½"	12"	7'-6"	½"	12"	13'-4"	1.205	79 94	12'-span
8"	7'	11"	9"	5/8"	6"	13'-4"	½"	12"	3'-0"	½"	12"	8'-9"	½"	12"	13'-4"	1.261	83.34	
8"	8'	11"	9"	5/8"	6"	13'-4"	½"	12"	3'-0"	½"	12"	9'-6"	½"	12"	13'-4"	1.316	85.04	
8"	9'	11"	9"	5/8"	6"	13-' 4"	½"	12"	3'-0"	½"	12"	10'-6"	½"	12"	13'-4"	1.372	88 44	
8"	10'	11"	9"	5/8"	6"	13'-4"	½"	12"	3'-0"	½"	12"	11'-6"	½"	12"	13'-4"	1.427	90.14	

1 Depth of fill.
2 Height of culvert.
3 Thickness of top.
4 Thickness of bottom and sides.

NOTE: This table is calculated for a minimum fill of 8 in. of rocks over culvert slabs and is correct for all fills up to 6 ft. in height.

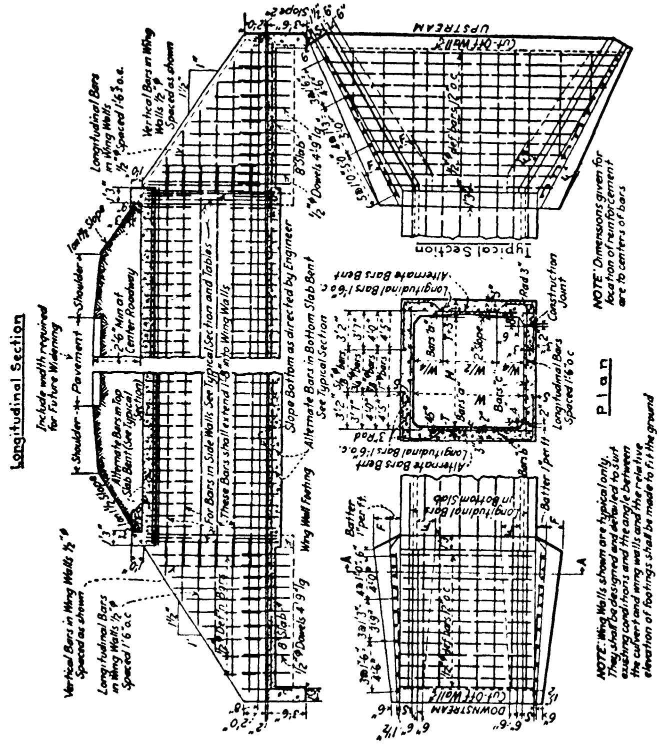
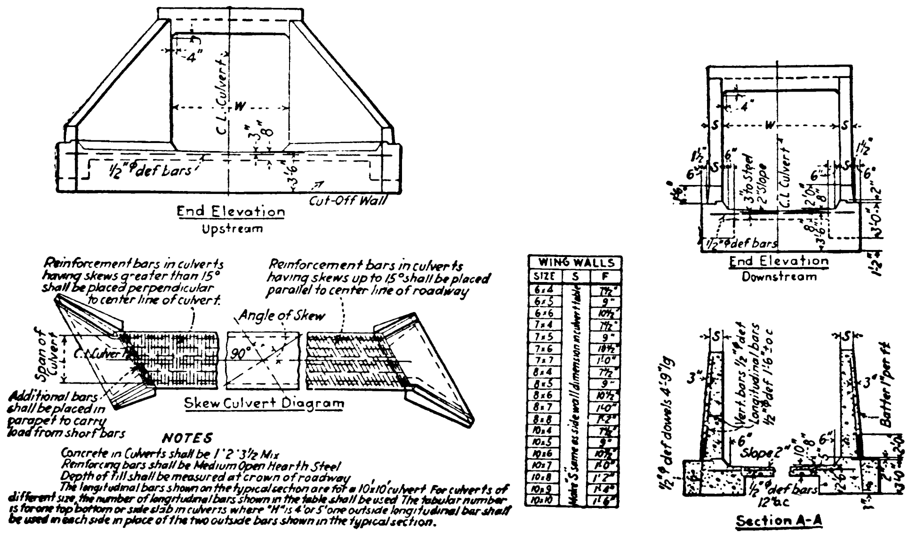


FIG. 67.—Reinforced box culverts N. Y. State, 1926.

Note.—Suitable on soft foundations or under very deep fills. Not a good design for firm soils under light fills.



WING WALLS

SIZE	S	F
6×4	Note "S" same as side wall dimension in culvert table	7½"
6×5		9"
6×6		10½"
7×4		7½"
7×5		9"
7×6		10½"
7×7		1'-0"
8×4		7½"
8×5		9"
8×6		10½"
8×7		1'-0"
8×8		1'-2"
10×4		7½"
10×5		9"
10×6		10½"
10×7		1'-0"
10×8		1'-2"
10×9		1'-4"
10×10		1'-6"

NOTES

Concrete in Culverts shall be 1 : 2 : 3½ Mix
Reinforcing bars shall be Medium Open Hearth Steel
Depth of fill shall be measured at crown of roadway
The longitudinal bars shown on the typical section are for a 10'×10' culvert. For culverts of different size, the number of longitudinal bars shown in the table shall be used. The tabular number is for one top bottom or side slab in culverts where "H" is 4' or 5' one outside longitudinal bar shall be used in each side in place of the two outside bars shown in the typical section.

FIG. 67.—(Continued.)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS

Inside culvert dimensions		Depth of fill over top of culverts												
		Between 2′-6″ and 8 ft. deep												
		Top & bottom slabs							Side slabs					
		Thickness, T	Reinforcement						Thickness, S	Reinforcement				
			Transverse				Longitudinal			Vertical			Longitudinal	
W	H		Size φ	Spacing	Length bars "a"	Length bars "b"	No.	Size φ		Size φ	Spacing	Length bars "c"	No.	Size φ
6′	4′	9″	5/8″	6″	7′-0″	14′-0″	7	1/2″	9″	1/2″	12″	5′-3″	4	1/2″
6′	5′	9″	5/8″	6″	7′-0″	14′-0″	7	1/2″	0″	1/2″	7½″	6′-3″	4	1/2″
6′	6′	9″	5/8″	6″	7′-0″	14′-0″	7	1/2″	9″	1/2″	5½″	7′-3″	6	1/2″
7′	4′	10″	3/4″	7½″	8′-2″	16′-3″	8	1/2″	10″	1/2″	12″	5′-5″	4	1/2″
7′	5′	10″	3/4″	7½″	8′-2″	16′-3″	8	1/2″	10″	1/2″	8½″	6′-5″	4	1/2″
7′	6′	10″	3/4″	7½″	8′-2″	16′-3″	8	1/2″	10″	1/2″	6″	7′-5″	6	1/2″
7′	7′	10″	3/4″	7½″	8′-2″	16′-3″	8	1/2″	10″	1/2″	4½″	8′-5″	6	1/2″
8′	4′	11″	3/4″	6″	9′-4″	17′-8″	9	1/2″	11″	1/2″	12″	5′-7″	4	1/2″
8′	5′	11″	3/4″	6″	9′-4″	17′-8″	9	1/2″	11″	1/2″	9½″	6′-7″	4	1/2″
8′	6′	11″	3/4″	6″	9′-4″	17′-8″	9	1/2″	11″	1/2″	6½″	7′-7″	6	1/2″
8′	7′	11″	3/4″	6″	9′-4″	17′-8″	9	1/2″	11″	1/2″	5″	8′-7″	6	1/2″
8′	8′	11″	3/4″	6″	9′-4″	17′-8″	9	1/2″	11″	5/8″	5½″	9′-7″	7	1/2″
10′	4′	14″	3/4″	5″	11′-8″	20′-9″	10	5/8″	13″	1/2″	12″	6′-1″	4	5/8″
10′	5′	14″	3/4″	5″	11′-8″	20′-9″	10	5/8″	13″	1/2″	11″	7′-1″	4	5/8″
10′	6′	14″	3/4″	5″	11′-8″	20′-9″	10	5/8″	13″	1/2″	7½″	8′-1″	6	5/8″
10′	7′	14″	3/4″	5″	11′-8″	20′-9″	10	5/8″	13″	5/8″	9″	9′-1″	6	5/8″
10′	8′	14″	3/4″	5″	11′-8″	20′-9″	10	5/8″	13″	5/8″	7″	10′-1″	7	5/8″
10′	9′	14″	3/4″	5″	11′-8″	20′-9″	10	5/8″	13″	5/8″	5½″	11′-1″	8	5/8″
10′	10′	14″	3/4″	5″	11′-8″	20′-9″	10	5/8″	13″	5/8″	4″	12′-1″	8	5/8″

FIG. 67.—(*Continued.*)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS.—(*Continued*)

Inside culvert dimensions		Depth of fill over top of culverts												
		9 ft. deep												
		Top & bottom slabs							Side slabs					
		Thickness, T	Reinforcement						Thickness, S	Reinforcement				
			Transverse				Longitudinal			Vertical			Longitudinal	
W	H		Size φ	Spacing	Length bars "a"	Length bars "b"	No.	Size φ		Size φ	Spacing	Length bars "c"	No.	Size φ
6'	4'	9"	5/8"	6"	7'-0"	14'-0"	7	1/2"	9"	1/2"	10"	5'-3"	4	1/2"
6'	5'	9"	5/8"	6"	7'-0"	14'-0"	7	1/2"	9"	1/2"	7"	6'-3"	4	1/2"
6'	6'	9"	5/8"	6"	7'-0"	14'-0"	7	1/2"	9"	1/2"	5"	7'-3"	6	1/2"
7'	4'	11"	3/4"	8"	8'-4"	16'-8"	8	1/2"	11"	1/2"	12"	5'-7"	4	1/2"
7'	5'	11"	3/4"	8"	8'-4"	16'-8"	8	1/2"	11"	1/2"	9"	6'-7"	4	1/2"
7'	6'	11"	3/4"	8"	8'-4"	16'-8"	8	1/2"	11"	1/2"	6"	7'-7"	6	1/2"
7'	7'	11"	3/4"	8"	8'-4"	16'-8"	8	1/2"	11"	1/2"	4 1/2"	8'-7"	6	1/2"
8'	4'	12"	3/4"	6 1/2"	9'-6"	18'-1"	9	1/2"	12"	1/2"	12"	5'-9"	4	1/2"
8'	5'	12"	3/4"	6 1/2"	9'-6"	18'-1"	9	1/2"	12"	1/2"	9 1/2"	6'-9"	4	1/2"
8'	6'	12"	3/4"	6 1/2"	9'-6"	18'-1"	9	1/2"	12"	1/2"	7"	7'-9"	6	1/2"
8'	7'	12"	3/4"	6 1/2"	9'-6"	18'-1"	9	1/2"	12"	5/8"	8"	8'-9"	6	1/2"
8'	8'	12"	3/4"	6 1/2"	9'-6"	18'-1"	9	1/2"	12"	5/8"	6"	9'-9"	7	1/2"
10'	4'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	1/2"	12"	6'-3"	4	5/8"
10'	5'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	1/2"	11"	7'-3"	4	5/8"
10'	6'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	1/2"	8"	8'-3"	6	5/8"
10'	7'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	5/8"	9"	9'-3"	6	5/8"
10'	8'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	5/8"	7"	10'-3"	7	5/8"
10'	9'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	5/8"	5 1/2"	11'-3"	8	5/8"
10'	10'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	5/8"	4 1/2"	12'-3"	8	5/8"

FIG. 67.—(*Continued*.)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS.—(*Continued*)

Inside culvert dimensions		Depth of fill over top of culverts												
		10 ft. deep												
		Top & bottom slabs							Side slabs					
		Thickness, T	Reinforcement						Thickness, S	Reinforcement				
			Transverse				Longitudinal			Vertical			Longitudinal	
W	H		Size ϕ	Spacing	Length bars "a"	Length bars "b"	No.	Size ϕ		Size ϕ	Spacing	Length bars "c"	No.	Size ϕ
6'	4'	10"	5/8"	6"	7'-2"	14'-5"	7	1/2"	10"	1/2"	11"	5'-5"	4	1/2"
6'	5'	10"	5/8"	6"	7'-2"	14'-5"	7	1/2"	10"	1/2"	7 1/2"	6'-5"	4	1/2"
6'	6'	10"	5/8"	6"	7'-2"	14'-5"	7	1/2"	10"	1/2"	5"	7'-5"	6	1/2"
7'	4'	11"	3/4"	7"	8'-4"	16'-8"	8	1/2"	11"	1/2"	12"	5'-7"	4	1/2"
7'	5'	11"	3/4"	7"	8'-4"	16'-8"	8	1/2"	11"	1/2"	8"	6'-7"	4	1/2"
7'	6'	11"	3/4"	7"	8'-4"	16'-8"	8	1/2"	11"	1/2"	5 1/2"	7'-7"	6	1/2"
7'	7'	11"	3/4"	7"	8'-4"	16'-8"	8	1/2"	11"	1/2"	4"	8'-7"	6	1/2"
8'	4'	12"	3/4"	6"	9'-6"	18'-1"	9	1/2"	12"	1/2"	12"	5'-9"	4	1/2"
8'	5'	12"	3/4"	6"	9'-6"	18'-1"	9	1/2"	12"	1/2"	9"	6'-9"	4	1/2"
8'	6'	12"	3/4"	6"	9'-6"	18'-1"	9	1/2"	12"	5/8"	10"	7'-9"	6	1/2"
8'	7'	12"	3/4"	6"	9'-6"	18'-1"	9	1/2"	12"	5/8"	7 1/2"	8'-9"	6	1/2"
8'	8'	12"	3/4"	6"	9'-6"	18'-1"	9	1/2"	12"	5/8"	5 1/2"	9'-9"	7	1/2"
10'	4'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	1/2"	12"	6'-3"	4	5/8"
10'	5'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	1/2"	10"	7'-3"	4	5/8"
10'	6'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	1/2"	7"	8'-3"	6	5/8"
10'	7'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	5/8"	8 1/2"	9'-3"	6	5/8"
10'	8'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	5/8"	6 1/2"	10'-3"	7	5/8"
10'	9'	15"	7/8"	6 1/2"	11'-10'	21'-11"	10	5/8"	14"	5/8"	5"	11'-3"	8	5/8"
10'	10'	15"	7/8"	6 1/2"	11'-10"	21'-11"	10	5/8"	14"	5/8"	4"	12'-3"	8	5/8"

FIG. 67.—(*Continued.*)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS.—(*Continued*)

Inside culvert dimensions		Depth of fill over top of culverts												
		11 ft. deep												
		Top & bottom slabs							Side slabs					
		Thickness, T	Reinforcement						Thickness, S	Reinforcement				
			Transverse				Longitudinal			Vertical			Longitudinal	
W	H		Size ϕ	Spacing	Length bars "a"	Length bars "b"	No.	Size ϕ		Size ϕ	Spacing	Length bars "c"	No.	Size ϕ
6′	4′	10″	5/8″	5½″	7′–2″	14′–5″	7	½″	10″	½″	10″	5′–5″	4	½″
6′	5′	10″	5/8″	5½″	7′–2″	14′–5″	7	½″	10″	½″	7″	6′–5″	4	½″
6′	6′	10″	5/8″	5½″	7′–2″	14′–5″	7	½″	10″	½″	4½″	7′–5″	6	½″
7′	4′	11″	¾″	6½″	8′–4″	16′–8″	8	½″	11″	½″	11½″	5′–7″	4	½″
7′	5′	11″	¾″	6½″	8′–4″	16′–8″	8	½″	11″	½″	7½″	6′–7″	4	½″
7′	6′	11″	¾″	6½″	8′–4″	16′–8″	8	½″	11″	½″	5½″	7′–7″	6	½″
7′	7′	11″	¾″	6½″	8′–4″	16′–8″	8	½″	11″	½″	4″	8′–7″	6	½″
8′	4′	13″	¾″	6″	9′–8″	18′–6″	8	5/8″	13″	½″	12″	5′–11″	4	5/8″
8′	5′	13″	¾″	6″	9′–8″	18′–6″	8	5/8″	13″	½″	9″	6′–11″	4	5/8″
8′	6′	13″	¾″	6″	9′–8″	18′–6″	8	5/8″	13″	5/8″	10″	7′–11″	6	5/8″
8′	7′	13″	¾″	6″	9′–8″	18′–8″	8	5/8″	13″	5/8″	7½″	8′–11″	6	5/8″
8′	8′	13″	¾″	6″	9′–8″	18′–6″	8	5/8″	13″	5/8″	5½″	9′–11″	7	5/8″
10′	4′	15″	7/8″	6″	11′–10″	21′–11″	10	5/8″	14″	½″	12″	6′–3″	4	5/8″
10′	5′	15″	7/8″	6″	11′–10″	21′–11″	10	5/8″	14″	½″	9½″	7′–3″	4	5/8″
10′	6′	15″	7/8″	6″	11′–10″	21′–11″	10	5/8″	14″	5/8″	10½″	8′–3″	6	5/8″
10′	7′	15″	7/8″	6″	11′–10″	21′–11″	10	5/8″	14″	5/8″	8″	9′–3″	6	5/8″
10′	8′	15″	7/8″	6″	11′–10″	21′–11″	10	5/8″	14″	5/8″	6″	10′–3″	7	5/8″
10′	9′	15″	7/8″	6″	11′–10″	21′–11″	10	5/8″	14″	5/8″	5″	11′–3″	8	5/8″
10′	10′	15″	7/8″	6″	11′–10″	21′–11″	10	5/8″	14″	5/8″	4″	12′–3″	8	5/8″

FIG. 67.—(*Continued.*)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS.—(*Continued*)

Inside culvert dimensions		Depth of fill over top of culverts												
		12 ft. deep												
		Top & bottom slabs							Side slabs					
		Thickness, T	Reinforcement						Thickness, S	Reinforcement				
			Transverse				Longitudinal			Vertical			Longitudinal	
W	H		Size ϕ	Spacing	Length bars "a"	Length bars "b"	No.	Size ϕ		Size ϕ	Spacing	Length bars "c"	No.	Size ϕ
6′	4′	10″	¾″	7½″	7′–2″	15′–3″	7	½″	10″	½″	9½″	5′–5″	4	½″
6′	5′	10″	¾″	7½″	7′–2″	15′–3″	7	½″	10″	½″	6½″	6′–5″	4	½″
6′	6′	10″	¾″	7½″	7′–2″	15′–3″	7	½″	10″	½″	4½″	7′–5″	6	½″
7′	4′	12″	¾″	6½″	8′–6″	17′–1″	8	½″	12″	½″	11½″	5′–9″	4	½″
7′	5′	12″	¾″	6½″	8′–6″	17′–1″	8	½″	12″	½″	8″	6′–9″	4	½″
7′	6′	12″	¾″	6½″	8′–6″	17′–1″	8	½″	12″	½″	5½″	7′–9″	6	½″
7′	7′	12″	¾″	6½″	8′–6″	17′–1″	8	½″	12″	½″	4″	8′–9″	6	½″
8′	4′	13″	¾″	5½″	9′–8″	18′–6″	8	⅝″	13″	½″	12″	5′–11″	4	⅝″
8′	5′	13″	¾″	5½″	9–8″	18′–6″	8	⅝″	13″	½″	8½″	6′–11″	4	⅝″
8′	6′	13″	¾″	5½″	9′–8″	18′–6″	8	⅝″	13″	⅝″	9½″	7′–11″	6	⅝″
8′	7′	13″	¾″	5½″	9′–8″	18′–6″	8	⅝″	13″	⅝″	7″	8′–11″	6	⅝″
8′	8′	13″	¾″	5½″	9′–8″	18′–6″	8	⅝″	13″	⅝″	5½″	9′–11″	7	⅝″
10′	4′	16″	⅞″	6″	12′–0″	22′–4″	10	⅝″	15″	½″	12″	6′–5″	4	⅝″
10′	5′	16″	⅞″	6″	12′–0″	22′–4″	10	⅝″	15″	½″	9½″	7′–5″	4	⅝″
10′	6′	16″	⅞″	6″	12′–0″	22′–4″	10	⅝″	15″	⅝″	10½″	8′–5″	6	⅝″
10′	7′	16″	⅞″	6″	12′–0″	22′–4″	10	⅝″	15″	⅝″	8″	9′–5″	6	⅝″
10′	8′	16″	⅞″	6″	12′–0″	22′–4″	10	⅝″	15″	⅝″	6″	10′–5″	7	⅝″
10′	9′	16″	⅞″	6″	12′–0″	22′–4″	10	⅝″	15″	⅝″	5″	11′–5″	8	⅝″
10′	10′	16″	⅞″	6″	12′–0″	22′–4″	10	⅝″	15″	⅝″	4″	12′–5″	8	⅝″

FIG. 67.—(*Continued.*)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS.—(*Continued*)

Inside culvert dimensions		Depth of fill over top of culverts												
		13 ft. deep												
		Top & bottom slabs							Side slabs					
		Thickness, T	Reinforcement						Thickness, S	Reinforcement				
			Transverse			Longitudinal				Vertical			Longitudinal	
W	H		Size ϕ	Spacing	Length bars "a"	Length bars "b"	No.	Size ϕ		Size ϕ	Spacing	Length bars "c"	No.	Size ϕ
6′	4′	11″	¾″	7½″	7′–2″	15′–6″	7	½″	10″	½″	9″	5′–7″	4	½″
6′	5′	11″	¾″	7½″	7′–2″	15′–6″	7	½″	10″	½″	6″	6′–7″	4	½″
6′	6′	11″	¾″	7½″	7′–2″	15′–6″	7	½″	10″	½″	4″	7′–7″	6	½″
7′	4′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	11″	5′–9″	4	½″
7′	5′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	7½″	6′–9″	4	½″
7′	6′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	5″	7′–9″	6	½″
7′	7′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	4″	8′–9″	6	½″
8′	4′	13″	¾″	5″	9′–8″	18′–6″	8	⅝″	13″	½″	12″	5′–11″	4	⅝″
8′	5′	13″	¾″	5″	9′–8″	18′–6″	8	⅝″	13″	½″	8″	6′–11″	4	⅝″
8′	6′	13″	¾″	5″	9′–8″	18′–6″	8	⅝″	13″	⅝″	9″	7′–11″	6	⅝″
8′	7′	13″	¾″	5″	9′–8″	18′–6″	8	⅝″	13″	⅝″	6½″	8′–11″	6	⅝″
8′	8′	13″	¾″	5″	9′–8″	18′–6″	8	⅝″	13″	⅝″	5″	9′–11″	7	⅝″
10′	4′	16″	1″	7½″	12′–0″	23′–2″	10	⅝″	15″	½″	12″	6′–5″	4	⅝″
10′	5′	16″	1″	7½″	12′–0″	23′–2″	10	⅝″	15″	½″	9″	7′–5″	4	⅝″
10′	6	16″	1″	7½″	12′–0″	23′–2″	10	⅝″	15″	⅝″	10″	8′–5″	6	⅝″
10′	7′	16″	1″	7½″	12′–0″	23′–2″	10	⅝″	15″	⅝″	7½″	9′–5″	6	⅝″
10′	8′	16″	1″	7½″	12′–0″	23′–2″	10	⅝″	15″	¾″	8½″	10′–5″	7	⅝″
10′	9′	16″	1″	7½″	12′–0″	23′–2″	10	⅝″	15″	¾″	6½″	11′–5″	8	⅝″
10′	10′	16″	1″	7½″	12′–0″	23′–2″	10	⅝″	15″	¾″	5½″	12′–5″	8	⅝″

FIG. 67.—(*Continued.*)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS.—(*Continued*)

Inside culvert dimensions		Depth of fill over top of culverts												
		14 ft. deep												
		Top & bottom slabs							Side slabs					
			Reinforcement							Reinforcement				
		Thickness, T	Transverse				Longitudinal		Thickness, S	Vertical			Longitudinal	
W	H		Size φ	Spacing	Length bars "a"	Length bars "b"	No.	Size φ		Size φ	Spacing	Length bars "c"	No.	Size φ
6′	4′	11″	¾″	7″	7′–2″	15′–6″	7	½″	10″	½″	8″	5′–7″	4	½″
6′	5′	11″	¾″	7″	7′–2″	15′–6″	7	½″	10″	½″	5½″	6′–7″	4	½″
6′	6′	11″	¾″	7″	7′–2″	15′–6″	7	½″	10″	½″	4″	7′–7″	6	½″
7′	4′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	10″	5′–9″	4	½″
7′	5′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	7″	6′–9″	4	½″
7′	6′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	5″	7′–9″	6	½″
7′	7′	12″	¾″	6″	8′–6″	17′–1″	8	½″	12″	½″	3½″	8′–9″	6	½″
8′	4′	14″	¾″	5″	9′–8″	18′–9″	8	⅝″	13″	½″	11″	6′–1″	4	⅝″
8′	5′	14″	¾″	5″	9′–8″	18′–9″	8	⅝″	13″	½″	7″	7′–1″	4	⅝″
8′	6′	14″	¾″	5″	9′–8″	18′–9″	8	⅝″	13″	⅝″	8″	8′–1″	6	⅝″
8′	7′	14″	¾″	5″	9′–8″	18′–9″	8	⅝″	13″	⅝″	6″	9′–1″	6	⅝″
8′	8′	14″	¾″	5″	9′–8″	18′–9″	8	⅝″	13″	⅝″	4½″	10′–1″	7	⅝″
10′	4′	17″	1″	7½″	12′–2″	23′–7″	10	⅝″	16″	½″	12″	6′–7′	4	⅝″
10′	5′	17″	1″	7½″	12′–2″	23′–7″	10	⅝″	16″	½″	9″	7′–7″	4	⅝″
10′	6′	17″	1″	7½″	12′–2″	23′–7″	10	⅝″	16″	⅝″	10″	8′–7″	6	⅝″
10′	7′	17″	1″	7½″	12′–2″	23′–7″	10	⅝″	16″	⅝″	7½″	9′–7″	6	⅝″
10′	8′	17″	1″	7½″	12′–2″	23′–7″	10	⅝″	16″	¾″	8½″	10′–7″	7	⅝″
10′	9′	17″	1″	7½″	12′–2″	23′–7″	10	⅝″	16″	¾″	6½″	11′–7″	8	⅝″
10′	10′	17″	1″	7½″	12′–2″	23′–7″	10	⅝″	16″	¾″	5½	12′–7″	8	⅝″

FIG. 67.—(*Continued.*)

TABLE OF SLAB THICKNESSES AND REINFORCEMENT FOR CULVERTS.—(*Continued*)

Inside culvert dimensions		Depth of fill over top of culverts												
		15 ft. deep												
		Top & bottom slabs							Side slabs					
		Thickness, T	Reinforcement						Thickness, S	Reinforcement				
			Transverse				Longitudinal			Vertical			Longitudinal	
W	H		Size ϕ	Spacing	Length bars "a"	Length bars "b"	No.	Size ϕ		Size ϕ	Spacing	Length bars "c"	No.	Size ϕ
6′	4′	11″	3/4″	6½″	7′-2″	15′-6″	7	1/2″	10″	1/2″	8″	5′-7″	4	1/2″
6′	5′	11″	3/4″	6½″	7′-2″	15′-6″	7	1/2″	10″	1/2″	5″	6′-7″	4	1/2″
6′	6′	11″	3/4″	6½″	7′-2″	15′-6″	7	1/2″	10″	1/2″	3½″	7′-7″	6	1/2″
7′	4′	13″	3/4″	6″	8′-6″	17′-4″	8	1/2″	12″	1/2″	9½″	5′-11″	4	1/2″
7′	5′	13″	3/4″	6″	8′-6″	17′-4″	8	1/2″	12″	1/2″	6½″	6′-11″	4	1/2″
7′	6′	13″	3/4″	6″	8′-6″	17′-4″	8	1/2″	12″	1/2″	4½″	7′-11″	6	1/2″
7′	7′	13″	3/4″	6″	8′-6″	17′-4″	8	1/2″	12″	1/2″	3½″	8′-11″	6	1/2″
8′	4′	14″	3/4″	5″	9′-8″	18′-9″	8	5/8″	13″	1/2″	10″	6′-1″	4	5/8″
8′	5′	14″	3/4″	5″	9′-8″	18′-9″	8	5/8″	13″	1/2″	7″	7′-1″	4	5/8″
8′	6′	14″	3/4″	5″	9′-8″	18′-9″	8	5/8″	13″	5/8″	8″	8′-1″	6	5/8″
8′	7′	14″	3/4″	5″	9′-8″	18′-9″	8	5/8″	13″	5/8″	6″	9′-1″	6	5/8″
8′	8′	14″	3/4″	5″	9′-8″	18′-9″	8	5/8″	13″	5/8″	4½″	10′-1″	7	5/8″
10′	4′	17″	1″	7″	12′-2″	23′-7″	10	5/8″	16″	1/2″	12″	6′-7″	4	5/8″
10′	5′	17″	1″	7″	12′-2″	23′-7″	10	5/8″	16″	1/2″	8½″	7′-7″	4	5/8″
10′	6′	17″	1″	7″	12′-2″	23′-7″	10	5/8″	16″	5/8″	9½″	8′-7″	6	5/8″
10′	7′	17″	1″	7″	12′-2″	23′-7″	10	5/8″	16″	5/8″	7″	9′-7″	6	5/8″
10′	8′	17″	1″	7″	12′-2″	23′-7″	10	5/8″	16″	3/4″	8″	10′-7″	7	5/8″
10′	9′	17″	1″	7″	12′-2″	23′-7″	10	5/8″	16″	3/4″	6½″	11′-7″	8	5/8″
10′	10′	17″	1″	7″	12′-2″	23′-7″	10	5/8″	16″	3/4″	5″	12′-7″	8	5/8″

FIG. 67.—(*Contiuned.*)

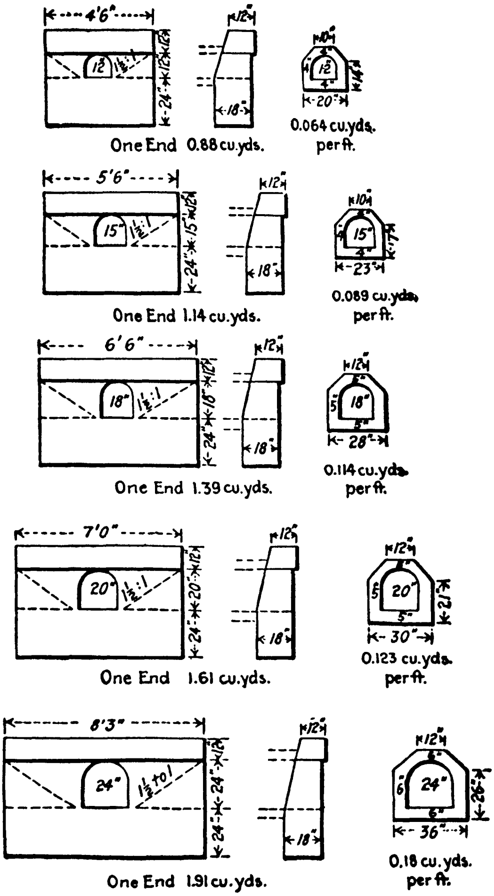


FIG. 68*A*.—Massachusetts standard for concrete arch culverts.

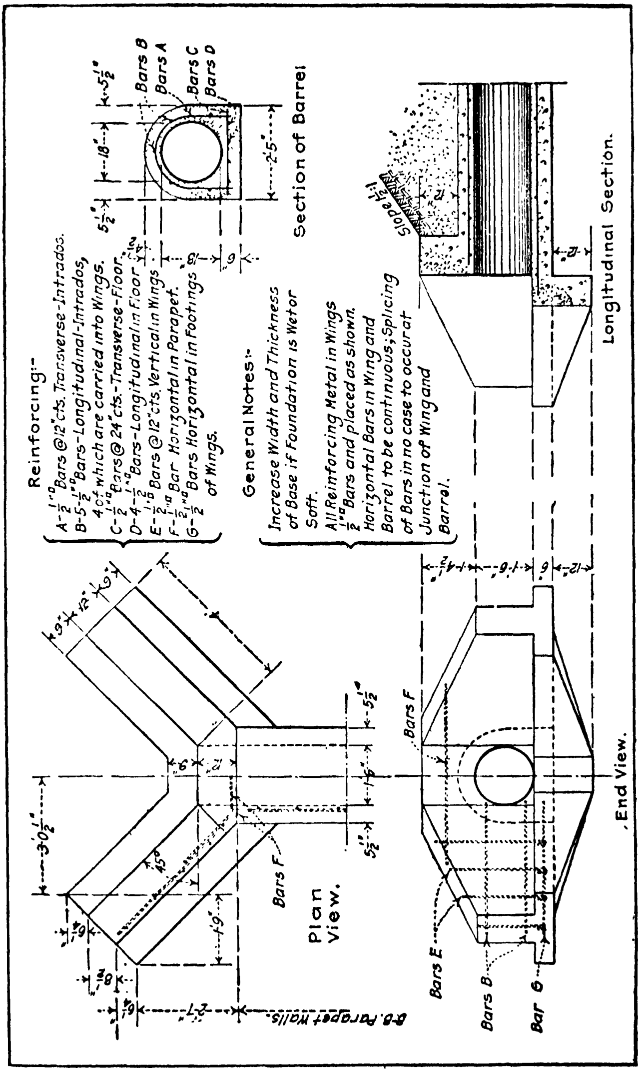


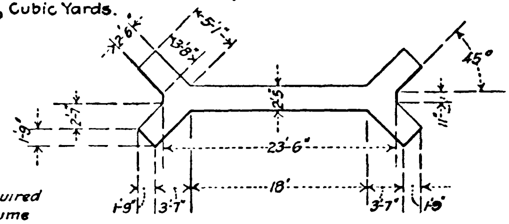
FIG. 68*B*.—State of Iowa 18" circular concrete culverts.

18 inch Circular Culvert.
Bill of Reinforcing Steel and Quantities of Concrete Materials
Cubic Contents of Culvert 20 ft. Back to Back of Parapet Walls Equals, $5\frac{4}{10}$ Cubic Yards.

Concrete Materials.

Item	Quantity Req. for Culvert 20 ft. b-b. Parapet Walls.	Additional Amount per Ft. Additional Length of Barrel
If Sand and Stone are Used.		
Sand	2.38 Cu. Yds.	0.055 Cu. Yds.
Stone	4.80 " "	0.112 " "
Cement	7.9 bbls. or 31.6 Sack	0.182 bbls. or 0.73 Sacks
If Gravel alone is Used.		
Gravel	6.2 Cu. Yds.	0.125 Cu. Yds.
Cement	10.25 bbls. or 41.0 Sacks	0.238 bbls. or 0.95 Sacks

Note { *The Table above gives Theoretical Quantities required To allow for Shrinkage add 20 Per Cent to the Volume of Sand and Stone or Gravel given in Table.*

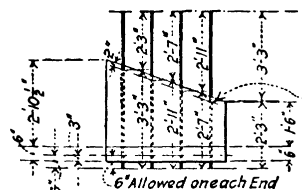


Staking Diagram.

Bill of Reinforcing for Culvert. 20 ft. b-b. Parapet Walls.

Mark	Position	No	Size	Length	Weight	Remarks
A	Transverse Barrel	22	½" Sq.	4'-10"	90 lb	
B	Longitud. Barrel	4	"	28'-6"	97 "	Continued into Wings
B	Longitud. Barrel	1	"	21'-6"	18 "	Center Bar
C	Transverse Floor	12	"	2'-3"	23 "	
D	Longitud. Floor	4	"	23'-0"	78 "	Cut in Field per Sketch
E	Vertical Wings	8	"	5'-6"	38 "	Continued into Wings
F	Horizont. Parapet	2	"	5'-6"	9 "	
G	Horizont. Footings	4	"	4'-0"	14 "	
	Total Reinforcing Steel				367 lb	

Note: { *For each additional Foot of Barrel use one additional Bar "A," One Half Bar "C," increase Length of Bars "B" and "D" by One Foot.*



Elevation of Wing.

FIG. 68B.—(*Continued.*)

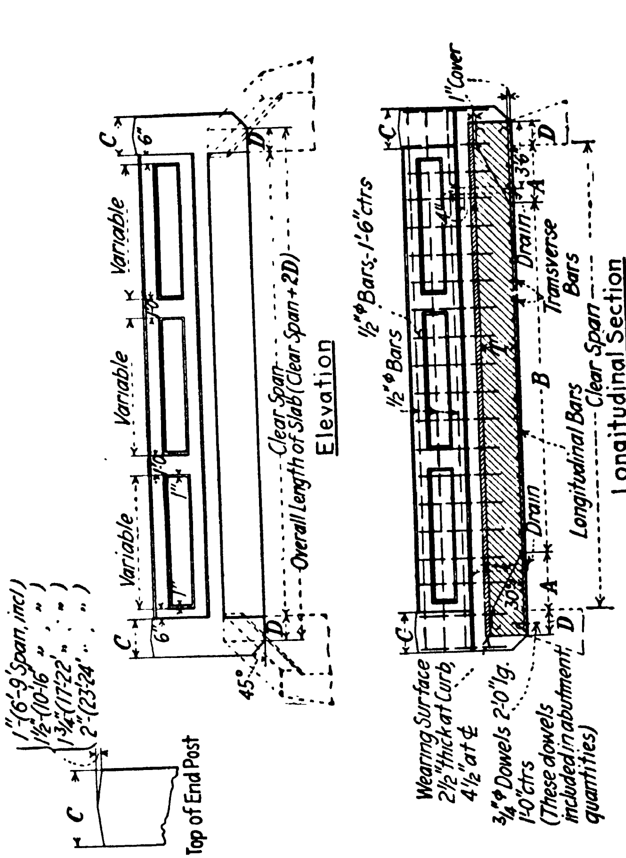


FIG. 69A.—Slab bridge superstructures (H-20 loading) (6 to 25 clear spans). New York State Standards 1926. Dimensions and bar lists see pages 239 and 240.

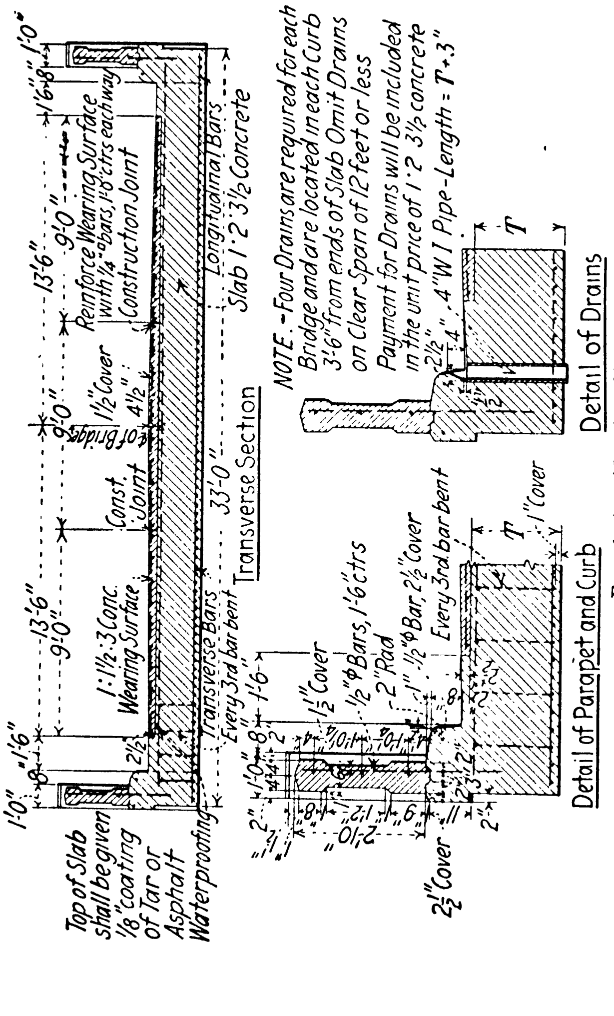


FIG. 69A.—(*Continued.*)

NOTE.—In 1927 pavement wearing surface has been increased in depth to 6" at middle and 4" at edges.

Dimensions						Slab									
						Longitudinal bars						Transverse bars			
Clear span	A	B	C	D	T	Size	C. to C.	No straight	Length	No bent	Length	Size	C. to C.	No.	Length
6	1′-0″	4′-0″	1′-0″	1′-0″	10″	½″□	4″	66	8′-2″	33	8′-5″	½″φ	1′-0″	11	32′-9″
7	1′-1″	4′-10″	1′-0″	1′-0″	10½″	⅝″φ	4½″	59	9′-4″	29	9′-7″	½″φ	1′-0″	12	32′-9″
8	1′-3″	5′-6″	1′-0″	1′-0″	11½″	⅝″φ	4″	66	10′-4″	33	10′-8″	½″φ	1′-0″	13	32′-9″
9	1′-4″	6′-4″	1′-0″	1′-0″	12″	¾″φ	5¼″	50	11′-6″	25	11′-10″	½″φ	1′-0″	14	32′-9″
10	1′-5″	7′-2″	1′-6″	1′-0″	12½″	¾″φ	5″	52	12′-6″	26	12′-10″	½″φ	1′-0″	15	32′-9″
11	1′-6″	8′-0″	1′-6″	1′-0″	13½″	¾″φ	4½″	59	13′-6″	29	13′-11″	½″φ	1′-0″	16	32′-9″
12	1′-8″	8′-8″	1′-6″	1′-3″	14½″	¾″φ	4¼″	62	15′-0″	31	15′-5″	½″φ	1′-0″	17	32′-9″
13	1′-9″	9′-6″	1′-6″	1′-3″	15″	¾″φ	4″	66	16′-0″	33	16′-6″	½″φ	1′-0″	18	32′-9″
14	1′-10″	10′-4″	1′-6″	1′-3″	15½″	¾″φ	4″	66	17′-0″	33	17′-6″	½″φ	1′-0″	19	32′-9″
15	1′-11″	11′-2″	1′-6″	1′-3″	16½″	⅞″φ	5¼″	50	18′-0″	25	18′-6″	½″φ	1′-0″	20	32′-9″
16	2′-0″	12′-0″	1′-6″	1′-3″	17″	⅞″φ	5″	52	19′-0″	26	19′-7″	½″φ	1′-0″	21	32′-9″
17	2′-2″	12′-8″	1′-9″	1′-3″	18″	⅞″φ	4¾″	55	20′-0″	28	20′-7″	½″φ	1′-0″	22	32′-9″
18	2′-4″	13′-4″	1′-9″	1′-6″	19″	⅞″φ	4½″	59	21′-6″	29	22′-2″	½″φ	1′-0″	23	32′-9″
19	2′-5″	14′-2″	1′-9″	1′-6″	19½″	⅞″φ	4¼″	62	22′-6″	31	23′-2″	½″φ	1′-0″	24	32′-9″
20	2′-6″	15′-0″	1′-9″	1′-6″	20½″	⅞″φ	4″	66	23′-6″	33	24′-3″	½″φ	1′-0″	25	32′-9″
21	2′-7″	15′-10″	1′-9″	1′-6″	21″	1″φ	5¼″	50	24′-8″	25	25′-5″	½″φ	1′-0″	26	32′-9″
22	2′-8″	16′-8″	1′-9″	1′-6″	21½″	1″φ	5″	52	25′-8″	26	26′-5″	½″φ	1′-0″	27	32′-9″
23	2′-10″	17′-4″	2′-0″	1′-6″	22½″	1″φ	4¾″	55	26′-8″	28	27′-6″	½″φ	1′-0″	28	32′-9″
24	2′-11′	18′-2″	2′-0″	1′-6″	23½″	1″φ	4½″	62	27′-8″	31	28′-6″	½″φ	1′-0″	29	32′-9″

FIG. 69A.—(*Continued.*)

Span	Parapet and curb						Wearing surface						Quantities				
	Longitudinal bars			Vertical bars			Longitudinal bars			Transverse bars			1-2-3½ concrete	1-1½-3 concrete	Cement	Waterproofing	Reinforcing bars
	No.	Size	Length	No.	Size	Length	No.	Size	Length	No.	Size	Length	Cu. yds.	Cu. yds.	Bbls.	Sq. ft.	Lbs.
6	10	½″φ	7′-8″	16	½″φ	4′-2″	30	¼″□	7′-8″	27	¼″□	8′-8″	9.2	3.6	22¾	219	1130
7	10	½″φ	8′-8″	18	½″φ	4′-2″	30	¼″□	8′-8″	30	¼″□	8′-8″	10.7	4.0	26¼	247	1340
8	10	½″φ	9′-8″	20	½″φ	4′-4″	30	¼″□	9′-8″	33	¼″□	8′-8″	13.0	4.4	31.0	274	1600
9	10	½″φ	10′-8″	20	½″φ	4′-4″	30	¼″□	10′-8″	36	¼″□	8′-8″	14.9	4.8	35.0	302	1880
10	10	½″φ	12′-8″	22	½″φ	4′-4″	30	¼″□	11′-8″	39	¼″□	8′-8″	17.0	5.4	39¾	329	2100
11	10	½″φ	13′-8″	24	½″φ	4′-4″	30	¼″□	12′-8″	42	¼″□	8′-8″	19.7	5.9	45¼	356	2470
12	10	½″φ	14′-8″	24	½″φ	4′-4″	30	¼″□	14′-2″	45	¼″□	8′-8″	23.5	6.5	53.0	398	2820
13	10	½″φ	15′-8″	26	½″φ	4′-4″	30	¼″□	15′-2″	48	¼″□	8′-8″	25.8	6.9	57¾	425	3160
14	10	½″φ	16′-8″	28	½″φ	4′-4″	60	¼″□	8′-6″	51	¼″□	8′-8″	28.3	7.3	62¾	452	3360
15	10	½″φ	17′-8″	28	½″φ	4′-4″	60	¼″□	9′-0″	54	¼″□	8′-8″	31.7	7.7	69½	480	3640
16	10	½″φ	18′-8″	30	½″φ	4′-4″	60	¼″□	9′-6″	57	¼″□	8′-8″	34.7	8.1	75¼	507	3960
17	10	½″φ	20′-2″	32	½″φ	4′-4″	60	¼″□	10′-0″	60	¼″□	8′-8″	38.6	8.6	82¾	535	4370
18	10	½″φ	21′-2″	32	½″φ	4′-4″	60	¼″□	10′-9″	63	¼″□	8′-8″	43.4	9.2	92¼	576	4900
19	10	½″φ	22′-2″	34	½″φ	4′-4″	60	¼″□	11′-3″	66	¼″□	8′-8″	46.6	9.6	98¼	603	5360
20	10	½″φ	23′-2″	36	½″φ	4′-4″	60	¼″□	11′-9″	69	¼″□	8′-8″	51.2	10.0	107.0	631	5890
21	10	½″φ	24′-2″	36	½″φ	4′-4″	60	¼″□	12′-3″	72	¼″□	8′-8″	54.6	10.4	113¾	658	6110
22	10	½″φ	25′-2″	38	½″φ	4′-4″	60	¼″□	12′-9″	75	¼″□	8′-8″	58.1	10.9	120¾	686	6570
23	10	½″φ	26′-8″	40	½″φ	4′-4″	60	¼″□	13′-3″	78	¼″□	8′-8″	63.1	11.4	130	713	7190
24	10	½″φ	27′-8″	40	½″φ	4′-4″	60	¼″□	13′-9″	81	¼″□	8′-8″	68.5	11.8	140	740	8200

Camber bridge 1/16″ per foot of clear span. Reinforcing bars may be spliced at places approved by the Engineer. Bars so spliced shall be lapped 40 diameters.

Concrete in slab and curb shall be nominal mix 1-2-3½. (2200 lbs. per sq. in.)

Concrete in parapet and concrete pavement shall be nominal mix 1-1½-3. (2500 lbs. per sq. in.)

Curbs and slab shall be poured at the same time, allowing no time for initial set to take place between them.

All reinforcing bars shall be medium, open hearth steel. Dimensions for bending reinforcing bars are measured from centers of bars. When reinforcing bars are hooked at end, the radius of the hook shall be 3 times the diameter of bar.

Parapet, fascia and curb surfaces shall be given a rubbed finish, cost to be included in unit price of 1-1½-3 concrete.

For name plate see sheet and the specifications.

Bridges of 12′-0″ clear span or less shall have one panel only on each face of parapet.

For details of abutments see page 241.

FIG. 69A.—(*Continued.*)

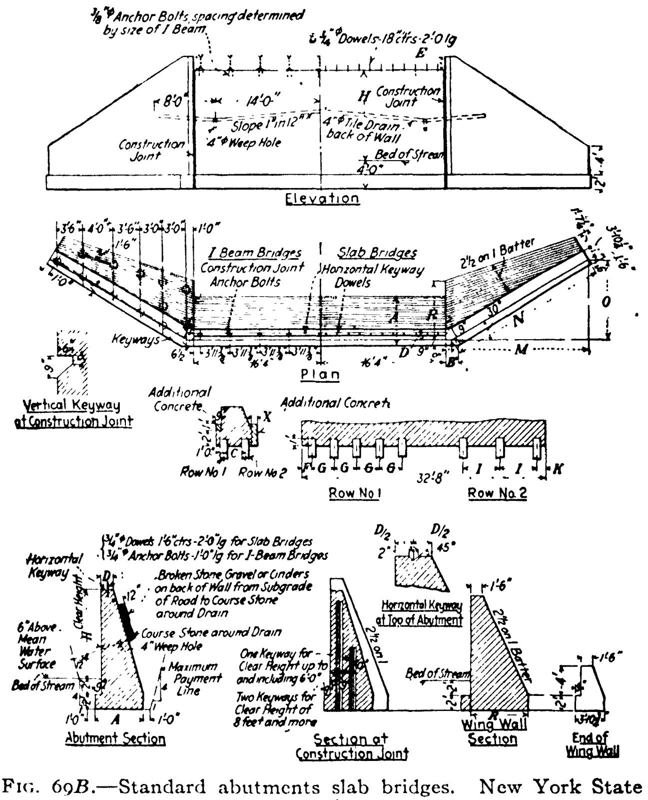


FIG. 69*B*.—Standard abutments slab bridges. New York State 1926.

(Dimensions and Quantities pages 242–245.)

NOTE.—Designed for ordinary soils. If gravel, hardpan or rock foundation occurs, wing back batters can be reduced.

Slab bridges													
		Abutment				Wing walls							
Clear span ft.	Clear height, H ft.	A	D	Cu. yds. conc. 1-abut.	Toe pres. tons	B	E	M	N	O	R	Cu. yds. conc. 2 wings
6 to 11	2	2′–9″	1′–0″	13.9	1.3	11″	10½″	1′–11½″	2′– 3″	1′– 1½″	3′– 5½″	3.8
	4	3′–3″		20.6	1.5	1′–2″		4′–11½″	5′– 8¾″	2′–10¼″	4′– 3¼″	10.7
	6	4′–0″		30.3	1.6	1′– 5″		7′–11½″	9′– 2″	4′– 7¼″	5′– 1″	20.1
	8	5′–0″		43.9	1.8	1′– 8″		10′–11½″	12′– 7¾″	6′– 4″	5′–10¾″	32.4
12 to 17	4	3′–3″	1′–3″	21.5	1.7	1′– 2″	1′– 3½″	5′– 7″	6′– 5½″	3′– 2¾″	4′– 5½″	12.2
	6	4′–0″		31.5	1.8	1′– 5″		8′– 7″	9′–11″	4′–11½″	5′– 3¼″	22.1
	8	4′–9″		43.3	2.0	1′– 8″		11′– 7″	13′– 4½″	6′– 8″	6′– 1″	35.1
	10	5′–6″		56.9	2.1	1′–11″		14′– 7″	16′–10″	8′– 4¾″	6′–10¾″	51.3
18 to 24	4	3′–3″	1′–6″	22.4	1.9	1′– 2″	1′–10½″	6′– 5½″	7′– 5¼″	3′– 8¾″	4′– 8″	14.2
	6	4′–0″		32.7	2.2	1′– 5″		9′– 5½″	10′–10¾″	5′– 5½″	5′– 6″	24.8
	8	4′–9″		44.8	2.4	1′– 8″		12′– 5½″	14′– 4¼″	7′– 2″	6′– 3½″	38.4
	10	5′–7″		59.5	2.5	1′–11″		15′– 5½″	17′– 9¾″	8′–11″	7′– 1½″	55.6
	12	6 –5″		76.3	2.5	2′– 2″		18′– 5½″	21′– 3½″	10′– 7¾″	7′–11¼″	76.5

FIG. 69B.—(*Continued.*)

Clear span, ft.	Clear height, ft.	Slab and I beam bridges								
		Abutment—pile foundation								
		C	Row No. 1		Row No. 2		No. piles required		Abutment footing	
			F	G	K	I			X	Additional concrete
6 to 11	2	Use box culvert, if soil bearing is too low.								Cu. yds.
	4	2′–0″	1′–4″	5′–0″	1′–4″	5′–0″	14	Staggered	9″	3.7
	6	2′–6″	1′–2″	4′–4″	3′–0″	6′–8″	13		6″	2.9
	8	2′–9″	1′–2″	4′–4″	1′–4″	6′–0″	14		0″	0.9
12 to 17	4	2′–0″	1′–4″	5′–0″	1′–4″	5′–0″	14	Staggered	9″	3.7
	6	2′–6″	1′–4″	3′–9″	3′–0″	6′–8″	14		6″	2.9
	8	2′–9″	1′–4″	3′–9″	1′–4″	6′–0″	15		0″	0.9
	10	3′–0″	1′–4″	3′–4″	1′–2″	4′–4″	18		0″	0.9
18 to 24	4	2′–0″	1′–4″	3′–4″	3′–0″	6′–8″	15	Staggered	9″	3.7
	6	2′–6″	1′–4″	3′–4″	3′–0″	6′–8″	15	Staggered	6″	2.9
	8	2′–9″	1′–4″	3′–0″	1′–4″	5′–0″	18		0″	0.9
	10	3′–0″	1′–4″	3′–0″	1′–4″	3′–9″	20		0″	0.9
	12	3′–6″	1′–4″	3′–0″	1′–4″	3′–0″	22		0″	0.9

NOTE: Estimated length of piles 20′.

FIG. 69B.—(*Continued.*)

Abutment—I beam bridges					Wing walls for I beam bridges—spans 10 to 24 incl.								
Clear span	Clear height, H	A	D	Cu. yds. conc. 1-abut	Clear span	Clear height, H	B	E	M	N	O	R	Cu. yds. conc. 2 wings
10′ to 24′	2′	2′–9″	1′–6″	14.5	10′ to 13′ incl. (12″ & 15″ beams)	2′	11″	1′–9″	3′– 3¼″	3′– 9¼″	1′–10¾″	3′– 9¾″	6.0
	4′	3′–3″	1′–6″	22.4		4′	1′– 2″	1′–9″	6′– 3¼″	7′– 3″	3′– 7½″	4′– 7¾″	13.6
	6′	4′–0″	1′–6″	32.7		6′	1′– 5″	1′–9″	9′– 3¼″	10′– 8½″	5′– 4¼″	5′– 5½″	23.9
	8′	4′–9″	1′–6″	44.8		8′	1′– 8″	1′–9″	12′– 3¼″	14′– 1¾″	7′– 1″	6′– 3¼″	37.3
	10′	5′–7″	1′–6″	59.5	14′ to 18′ incl. and 23′ (18″ beam)	4′	1′– 2″	2′–0″	6′– 7¾″	7′– 8″	3′–10″	4′– 9″	14.7
	12′	6 –5″	1′–6″	76.3		6′	1′– 5″	2′–0″	9′– 7¾″	11′– 1½″	5′– 6¾″	5′– 6¾″	25.5
						8′	1′– 8″	2′–0″	12′– 7¾″	14′– 7¼″	7′– 3½″	6′– 4½″	39.4
						10′	1′–11″	2′–0″	15′– 7¾″	18′– 1″	9′– 0½″	7′– 2¼″	56.3
					19′ to 22′ incl. and 24′ (20″ beam)	4′	1′– 2″	2′–2″	6′–10¾″	7′–11½″	3′–11¾″	4′– 9¾″	15.5
						6′	1′– 5″	2′–2″	9′–10¾″	11′– 5″	5′– 8¾″	5′– 7½″	26.5
						8′	1′– 8″	2′–2″	12′–10¾″	14′–10¾″	7′– 5½″	6′– 5¼″	40.5
						10′	1′–11″	2′–2″	15′–10¾″	18′– 4¼″	9′– 2¼″	7′– 3″	58.0
						12′	2′– 2″	2′–2″	18′–10¾″	21′– 9½″	10′–11″	8′– 0¾″	82.1
					23′ and 24′ (24″ beam)	4′	1′– 2″	2′–6″	7′– 4¾″	8′– 6½″	4′– 3¼″	4′–11¼″	17.1
						6′	1′– 5″	2′–6″	10′– 4¾″	12′– 0″	6′– 0″	5′– 9″	28.5
						8′	1′– 8″	2′–6″	13′– 4¾″	15′– 5¾″	7′– 9″	6′– 6¾″	41.6
						10′	1′–11″	2′–6″	16′– 4¾″	18′–11″	9′– 5½″	7′– 4¾″	61.2
						12′	2′– 2″	2′–6″	19′– 4¾″	22′– 4¾″	11′– 2½″	8′– 2½″	83.3

FIG. 69B.—I beam bridge abutments 13′ to 25′ spans.—(*Continued.*)

The abutments and wing walls shown on this drawing are typical only. The Engineer will give definite elevations and dimensions for each bridge.

The depth of footings shall be determined with respect to the character of the foundation material and the possibility of undermining—all footings shall rest on a firm foundation and except where rock is encountered, shall be at a depth at least four feet below the bed of stream or surface of ground.

The wing walls shall be designed and detailed to suit existing conditions and the angle between abutment and wing wall and the relative elevations of footings, shall be made to fit the ground. In case it appears that the soil will not safely withstand the unit pressure noted in the above table of abutment dimensions, timber piles shall be used and spaced as shown in the table. The piles are designed for a maximum load of 15 tons per pile. The tops of piles shall be placed below low water elevation. In case excessive erosive action of the stream is expected, piles shall be used rip rap placed in front of the abutment or other approved means taken to prevent erosion.

Concrete in abutments and wing walls shall be 1 : 2½ : 5 Mix, Item No. 21.

Keyways between abutments and wing walls and at all horizontal joints, shall comprise about 30 % of the area of surface.

All exposed edges of concrete shall be chamfered 1 inch.

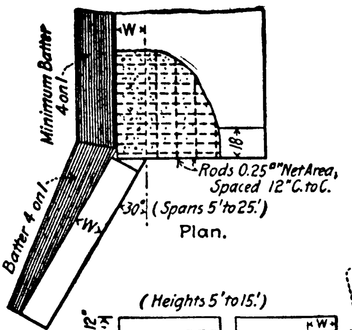
The bases of structures shown on this sheet shall be considered as approximate only and may be ordered in writing by the Engineer to be at any elevation and of any dimensions, necessary to give a proper foundation.

Payment for furnishing and placing dowels, expansion plates and bolts will be made at the respective contract prices for Metal Reinforcement and Structural Steel. (See superstructure details.)

Cost of furnishing and placing material for 4" tile drain on the back of abutments will be paid for under Item No. 7.

Porous material placed on back of wall will be paid for as excavation.

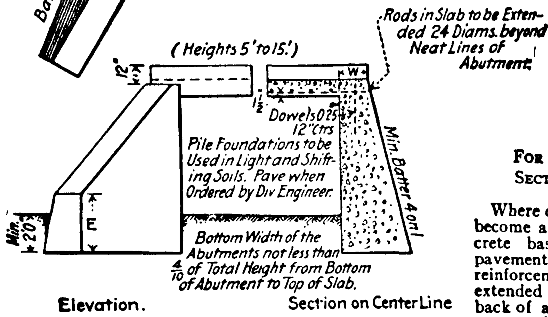
FIG. 69*B*.—(*Continued.*)



Note

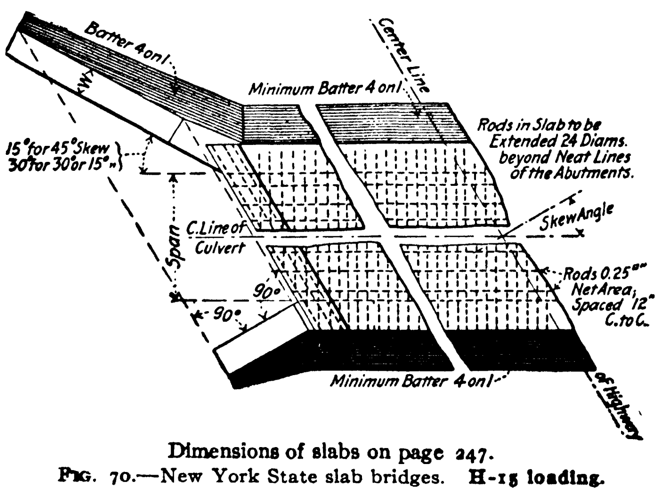
All rods to have a deformed cross-section. All rib metal to be of medium steel.

2d class concrete in all slabs and parapets. 3d class concrete in wings invert and abutments. Wing walls on the outlet end of all square culverts with concrete floors to be built parallel to the center line of the culvert. Round all exposed edges to 1½ inch radius.



For Typical Section "F"

Where culvert covers become a part of concrete base for brick pavement, transverse reinforcement should be extended 12" beyond back of abutment into concrete base.



Dimensions of slabs on page 247.

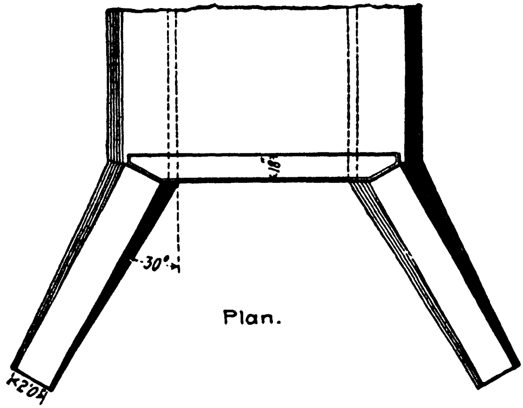
Fig. 70.—New York State slab bridges. **H-15 loading.**

Span	Thickness of Slab*	Net Area of Rods	Rod Spacing C–C	Length of Dowels
5	8″	0.25sq.″	$4\frac{1}{2}$″	12″
6	9″	"	4″	"
7	10″	0.39sq.″	$5\frac{3}{4}$″	"
8	10″	"	$5\frac{1}{4}$″	"
9	11″	"	5″	"
10	12″	"	$4\frac{3}{4}$″	"
11	12″	0.56sq.″	$6\frac{1}{4}$″	"
12	13″	"	6″	18″
13	13″	"	$5\frac{3}{4}$″	"
14	14″	"	$5\frac{3}{8}$″	"
15	14″	"	5″	"
16	15″	"	$4\frac{3}{4}$″	"
17	15″	"	$4\frac{3}{4}$″	"
18	16″	"	$4\frac{1}{2}$″	"
19	17″	"	$4\frac{1}{4}$″	"
20	18″	0.77sq.″	$5\frac{1}{4}$″	"
21	18″	"	$5\frac{1}{4}$″	"
22	19″	"	5″	24″
23	19″	"	5″	"
24	20″	"	$4\frac{5}{8}$″	"
25	21″	1.00sq.″	$5\frac{7}{8}$″	"

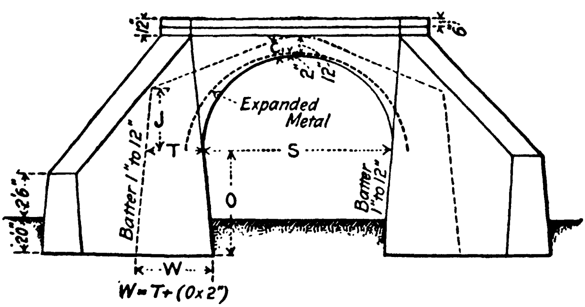
For Spans 5′ to 19′ W = 18″ For Clear Height 10′ or less
" " 5′ to 19′ W = 24″ " " " 11′ to 15′
" " 20′ to 25′ W = 24″ " " " 15′ or less
For Clear Height 7′ or less E = 3′-0″
" " " 8′ to 10′ E = 4′-0″
" " " above 10′ E = 5′-0″

* NOTE.—The thickness of slab given is for shallow fills. For the effect of deep fills see Table 193, page 1075.

FIG. 70.—(*Continued.*)



Plan.

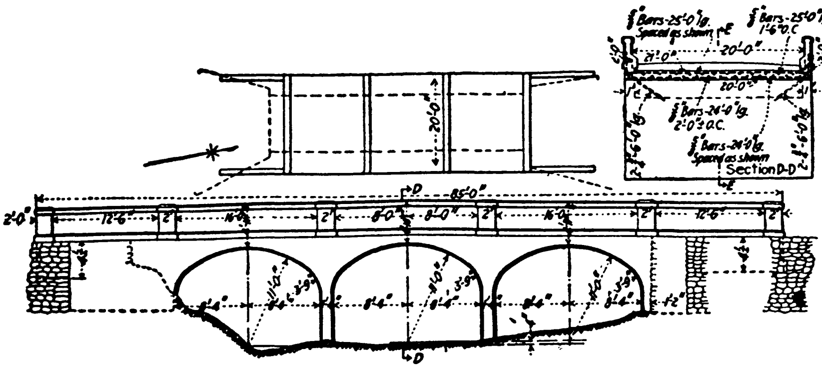


End Elevation.

GENERAL DIMENSIONS SEMI-CIRCULAR ARCH CULVERTS

S Span	Thickness at Springing Line		Thickness of Ring		Height of Haunch	
	T Concrete	K Masonry	C Concrete	R Masonry	J Concrete	V Masonry
6	2'-6"	2'-6"	10"	10"	1'-9"	2'-0"
8	2'-6"	2'-6"	11"	12"	2'-6"	2'-6"
10	3'-0"	3'-0"	12"	12"	3'-0"	3'-0"
12	3'-6"	3'-6"	14"	15"	3'-6"	3'-9"
14	3'-9"	3'-9"	15"	15"	4'-0"	4'-6"
16	4'-0"	4'-0"	16"	15"	4'-8"	5'-0"
18	4'-6"	4'-6"	18"	18"	5'-0"	5'-6"
20	5'-0"	5'-0"	18"	18"	5'-6"	6'-0"

FIG. 71.



STEEL LIST

Number Used	Size	Length	Position	Location
36	5/8	24'-0"	[illegible]	Inside Arch
30	"	25'-0"	"	Outside Arch
15	"	27'-0"	"	" " N
24	"	24'-0"	Hor.	Inside Arch
18	"	25'-0"	"	Outside Arch
8	3/4	7'-10"	Ver.	Inside End Posts
6	"	11'-6"	"	" Post
20	"	6'-6"	"	" "
16	"	11'-6"	"	Outside Wall
12	"	6'-8"	"	" "
12	"	5'-6"	"	" "
12	"	4'-10"	"	" "
6	"	4'-9"	"	" "
44	1/2	5'-4"	"	Inside Parapet
16	3/4	22'-0"	Hor.	Top of Parapet
8	3/4	22'-0"	"	Outside "
8	3/4	22'-0"	"	Wheel Guard
8	3/4	22'-0"	"	Outside [illegible]
4	3/4	20'-6"	"	" Wall
8	"	17'-0"	"	" "
2	"	11'-6"	"	" "
2	"	4'-0"	"	" "
4	"	5'-0"	Ver.	Abutment
16	"	8'-0"	"	Piers
12	"	21'-2"	Hor.	Piers & Abut's
8	"	6'-0"	Inclined	Buttresses
24	"	8'-0"	Ver	Posts
24	"	See Detail	Hor.	Posts, 6" c. to c.
4	"	15'-0"	"	Walls, West

FIG. 71A.—Typical multiple span. Reinforced concrete arch.

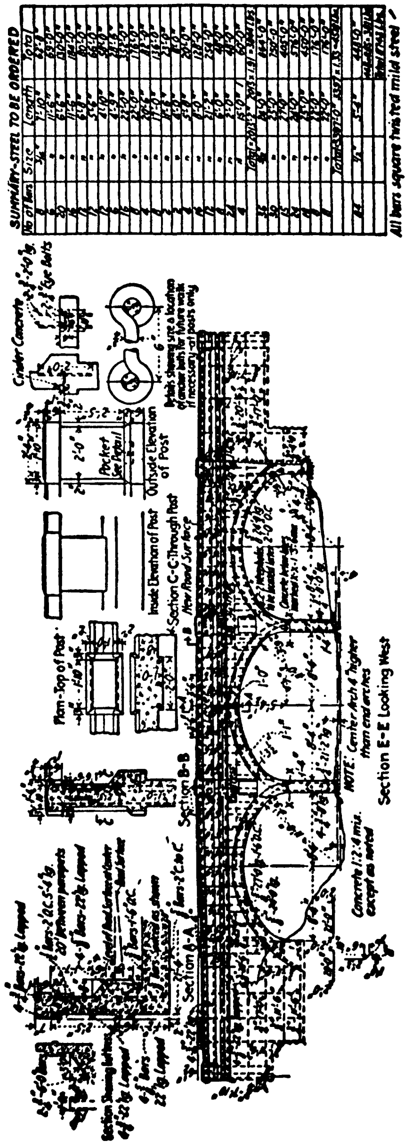
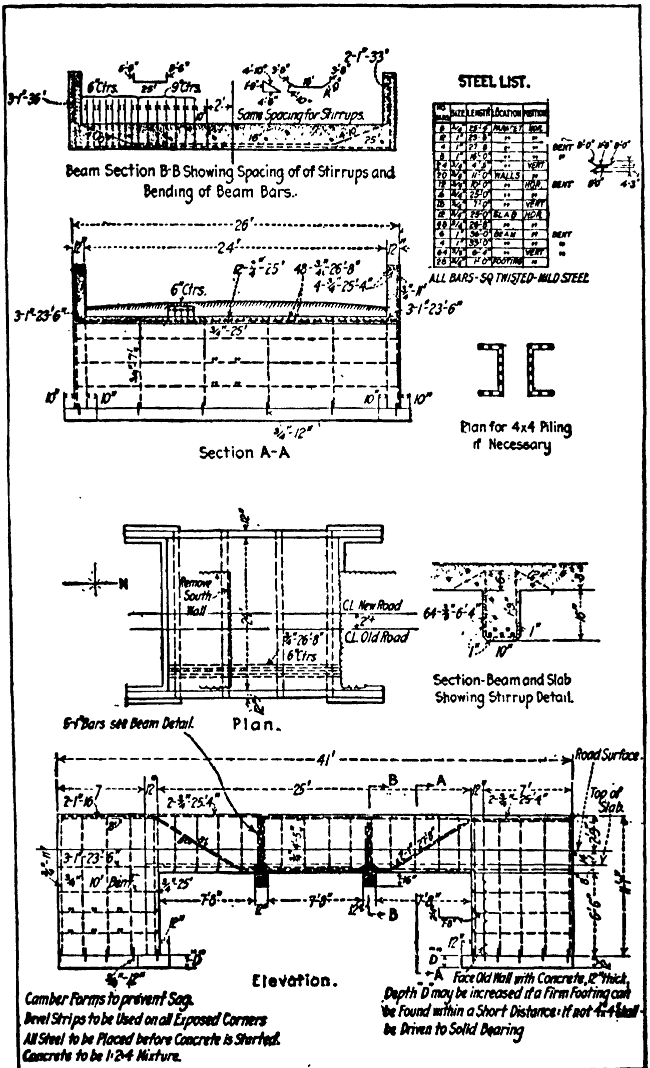


FIG. 71A.—(*Continued.*)



Design used by Monroe County, New York State.

FIG. 72.—Parapet girder bridge.

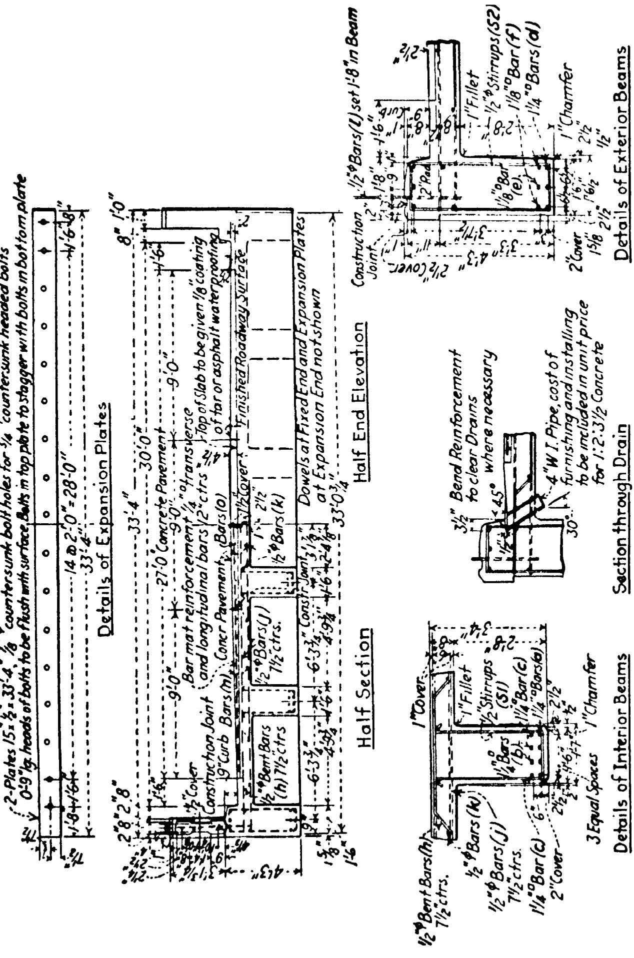
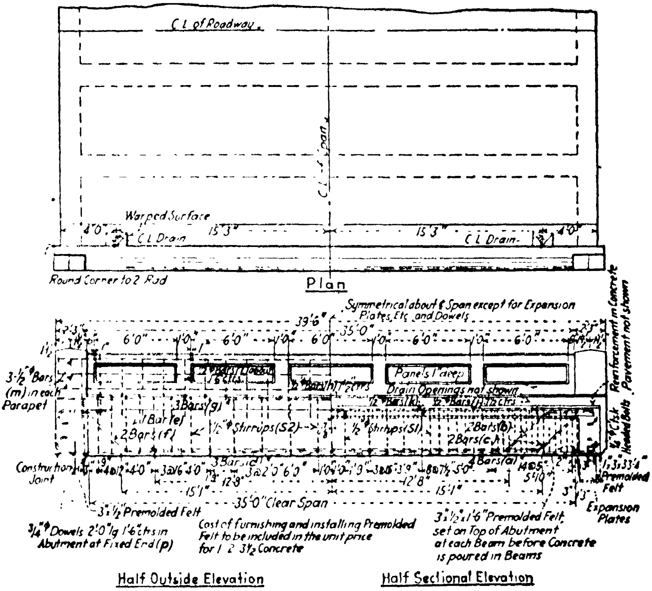


FIG. 73A.—N. Y. State. Typical T beam bridge. H-20 loading. (35 span).
For general dimensions, different spans, see page 255.



NOTES

Camber bridge 1/16" per foot of bridge span.

The above bar list makes no provision for splicing reinforcing bars except longitudinal mat bars. The ½"φ reinforcing bars may be spliced at places approved by the Engineer. Bars so spliced shall be lapped 40 diameters, and spaced to allow 1" of concrete between surfaces.

All concrete except in concrete pavement to be "Nominal Mix 1:2:3½–2200 #/□".

Concrete in wearing surface to be "Nominal Mix 1:1½:3–2500 #/□".

No construction joints other than shown on the plans will be permitted. The construction joint on the center line of roadway will be permitted only when approved by the Engineer in writing. Construction joints to be free from laitance.

The lower expansion plate shall be placed on the top of the abutment before the concrete in the abutment has set.

A layer of graphite axle grease about ⅛" thick shall be placed between the expansion plates before the concrete is poured in the beam-stems.

Curbs, slab and beam-stems must be poured at the same time, allowing no time for initial set to take place between them.

All reinforcing bars shall be of medium open hearth steel. All dimensions for bending reinforcing bars except as noted are center to center of bars.

All parapet, fascia and curb surfaces shall be given a rubbed finish, cost to be included in unit price for 1:1½:3 concrete.

For name plate, see sheet and specifications.

FIG. 73A.—(*Continued.*)

MATERIAL REQUIRED (no overrun or rounding)	
79.2 Cu. Yds 1:2:3½ Concrete	16.7 Cu Yds. 1 1½ 3 Concrete
168¼ Bbls. Cement	1740 Lb. Structural Steel
17,190 Lb. Metal Reinforcement	1056 Sq Ft Waterproofing

BAR LIST

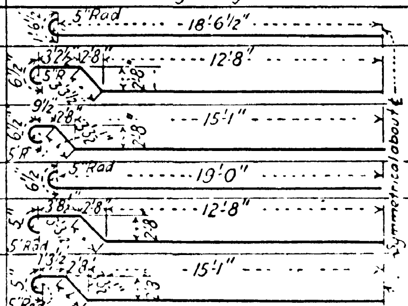
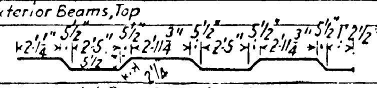
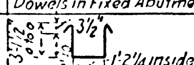
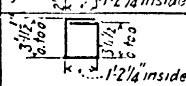
Mark	Number	Size	Length	Bending Diagram
a	16	1¼"□	40'-2"	
b	8	1¼"□	42'-5"	
c	8	1¼"□	42'-5"	
d	6	1¼"□	41'-1"	
e	2	1⅛"□	43'-2"	
f	4	1⅛"□	43'-2"	
g	6	1"□	39'-0"	Exterior Beams, Top
h	57	½"φ	34'-8"	
j	114	½"φ	32'-10"	Transverse Slab Bars, Top and Bottom
k	27	½"φ	38'-0"	Longitudinal Slab Bars, Top and Bottom
l	54	½"φ	4'-4"	Vertical Bars in Curbs and Parapet
m	6	½"φ	39'-2"	Longitudinal Bars in Parapet
n	117	¼"φ	8'-6"	Bar Mat. Reinforcement Transverse Bars
o	81	¼"φ	13'-4"	Bar Mat. Reinforcement Longitudinal Bars
p	22	¾"φ	2'-0"	Dowels in Fixed Abutments
S1	224	½"φ	8'-2"	 Interior Stringers
S2	52	½"φ	11'-6"	 Exterior Stringers

FIG. 73A.—Bar list 35 ft. clear span.—(*Continued.*)

TABLE OF DIMENSIONS AND REINFORCEMENT N. Y. STATE STANDARD REINFORCED CONCRETE STRINGER BEAM BRIDGES H-20 LOADING 30 FT. ROADWAY

Clear span between abutments	Number T beams	C to C T beams, inches	Depth T beam below floor slab, inches	Width T beam, inches	Depth floor slab (exclusive of pavement concrete)	Reinforcement						
						T beam (each)				Slab (long. bar)		
						Number long. bars	Size	Stirrups number	Size	Size	Spacing	
											Bottom	Top
25	6	6′ 03¾″	1′ 07″	1′ 06″	8″	8	1¼″ □	52	½″ φ	½″ φ	3¾″	7½″
26	6	6′ 03¾″	1′ 08″	1′ 06″	8″	8	1¼″ □	50	½″ φ	½″ φ	3¾″	7½″
28	6	6′ 03¾″	1′ 10″	1′ 06″	8″	8	1¼″ □	50	½″ φ	½″ φ	3¾″	7½″
30	6	6′ 03¾″	2′ 00″	1′ 06″	8″	8	1¼″ □	50	½″ φ	½″ φ	3¾″	7½″
32	6	6′ 03¾″	2′ 04″	1′ 06″	8″	8	1¼″ □	50	½″ φ	½″ φ	3¾″	7½″
34	6	6′ 03¾″	2 07″	1′ 06″	8″	8	1¼″ □	50	½″ φ	½″ φ	3¾″	7½″
35	6	6′ 03¾″	2′ 08″	1′ 06″	8″	8	1¼″ □	50	½″ φ	½″ φ	3¾″	7½″
36	6	6′ 03¾″	2′ 10″	1′ 06″	8″	8	1¼″ □	50	½″ φ	½″ φ	3¾″	7½″
38	6	6′ 03¾″	3′ 02″	1′ 06″	8″	8	1¼″ □	46	½″ φ	½″ φ	3¾″	7½″
40	6	6′ 03¾″	3′ 06″	1′ 06″	8″	8	1¼″ □	48	½″ φ	½″ φ	3¾″	7½″
42	6	6′ 03¾″	3′ 10″	1′ 06″	8″	8	1¼″ □	44	½″ φ	½″ φ	3¾″	7½″

FIG. 73A.—(*Continued.*)

QUANTITY FOR BEAM BRIDGES

Span	Metal reinforcement, pounds	Structural steel, pounds	Cement, barrels	1:2:3½ concrete, cubic yards	1:1½:3 concrete, cubic yards	Water-proofing, square feet	Estimated cost 1925	Slab transverse bars		
								Size	Spacing	
									Bottom	Top
25	12,200	1,400	102	45.5	12.3	768	$2,650			
26	12,630	1,400	107	48.1	12.7	796	2,770			
28	13,720	1,400	117	53.2	13.5	850	3,020			
30	14,630	1,400	129	58.7	14.3	905	3,255			
32	15,590	1,740	148	68.6	15.4	974	3,680			
34	16,670	1,740	162	76.0	16.3	1,029	3,090			
35	17,190	1,740	168	79.2	16.7	1,056	4,035	½″ φ	See plan	See plan
36	17,550	1,740	177	83.7	17.1	1,084	4,310			
38	18,450	2,080	199	95.7	18.2	1,152	4,795			
40	19,700	2,080	218	105.7	19.1	1,207	5,200			
42	20,640	2,080	237	116.2	19.8	1,262	$5,615			

FIG. 73A.—(*Continued.*)

Quantities in Concrete Deck Girder Highway Bridges. U. S. Bureau of Public Roads.

Span, ft.	16-ft. roadway		18-ft. roadway		20-ft. roadway		24-ft. roadway	
	Concrete, cu. yd.	Steel, lb.	Concrete, cu. yd.	Steel, lb.	Concrete, cu. yd.	Steel, lb.	Concrete, cu. yd.	Steel, lb.
16	14.3	2,820	15.3	3,160	16.5	3,510	19.2	3,980
18	16.0	3,130	17.7	3,520	19.0	3,880	22.2	4,430
20	17.8	3,770	19.3	4,330	21.2	4,760	24.2	5,470
22	20.1	4,100	21.8	4,690	23.3	5,140	27.4	5,930
24	22.3	4,460	23.6	5,710	25.7	6,230	29.6	7,220
26	23.8	5,260	25.7	6,140	28.0	6,670	32.4	7,770
28	29.1	5,870	30.2	7,070	32.8	7,440	37.7	8,940
30	32.6	6,260	34.1	7,570	36.9	7,980	42.5	9,580
32	34.6	7,560	38.1	8,100	41.2	8,500	47.6	10,240
34	38.3	8,100	42.5	8,580	43.5	10,180	53.0	10,850
36	42.2	8,550	45.5	10,300	47.9	10,770	56.6	13,040
38	46.4	9,070	50.1	10,910	52.8	11,420	62.5	13,810
40	50.8	9,540	55.0	11,520	57.8	12,030	68.6	14,600

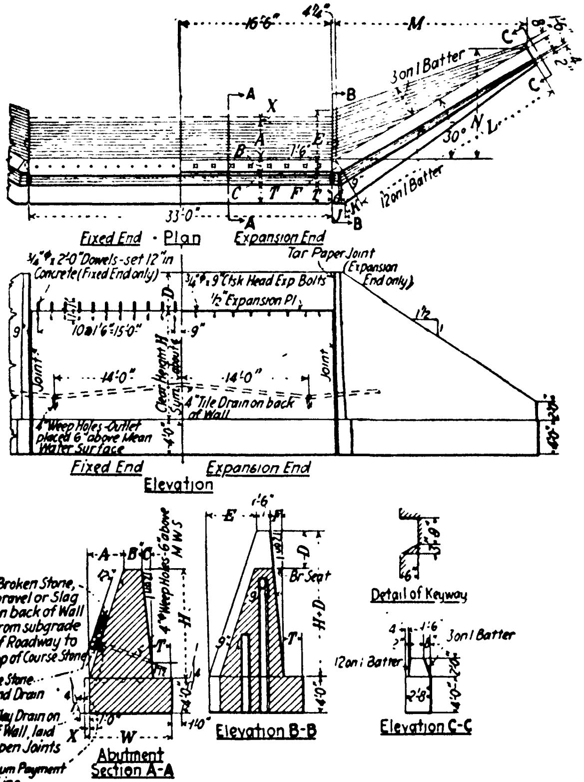
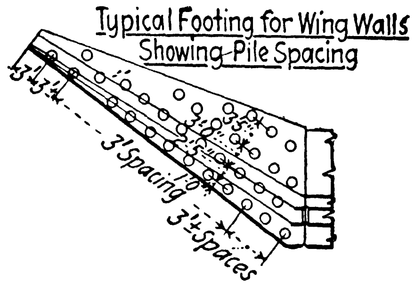


FIG. 73*B*.—Standard abutments for T beam bridges. New York State 1926. (Dimensions pages 260–264.)

NOTE.—Designed for ordinary soils. Base width can be reduced on gravel, hardpan or rock.



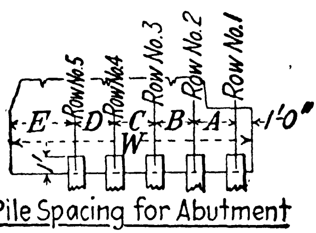


FIG. 73B.—(*Continued.*)

Clear span, feet	H	Abutment							
		A	B	C	Footing W	Footing T	Footing X	Cu. yds. concrete 1-2½-5	Toe pressure, tons sq. ft.
25 and 26	8	3′- 0″	1′-6″	8″	6′- 8″	1′- 6″	0	65.2	1.9
	10	3′-10″	1′-6″	10″	7′- 9″	1′- 7″	0	84.6	2.2
	12	4′- 8″	1′-6″	1′-0″	8′-10″	1′- 8″	0	106.2	2.3
	14	5′- 6″	1′-6″	1′-2″	9′-11″	1′- 9″	0	131.2	2.5
	16	6′- 4″	1′-6″	1′-4″	10′-11″	1′- 9″	0	157.5	2.7
28	8	3′- 0″	1′-6″	8″	6′- 8″	1′- 6″	0	65.2	2.1
	10	3′-10″	1′-6″	10″	7′- 9″	1′- 7″	0	84.6	2.3
	12	4′- 8″	1′-6″	1′-0″	8′-10″	1′- 8″	0	106.8	2.4
	14	5′- 6″	1′-6″	1′-2″	9′-11″	1′- 9″	0	131.2	2.6
	16	6′- 4″	1′-6″	1′-4″	10′-11″	1′- 9″	0	157.5	2.8
30	8	3′- 0″	1′-6″	8″	6′- 8″	1′- 6″	0	65.2	2.2
	10	3′-10″	1′-6″	10″	7′- 9″	1′- 7″	0	84.6	2.3
	12	4′- 8″	1′-6″	1′-0″	8′-10″	1′- 8″	0	106.8	2.5
	14	5′- 6″	1′-6″	1′-2″	9′-11″	1′- 9″	0	131.2	2.6
	16	6′- 4″	1′-6″	1′-4″	10′-11″	1′- 9″	0	157.5	2.8
32	10	3′- 5″	1′-9″	10″	7′- 8″	1′- 8″	0	85.0	2.4
	12	4′- 3″	1′-9″	1′-0″	8′- 9″	1′- 9″	0	106.9	2.6
	14	5′- 2″	1′-9″	1′-2″	9′-11″	1′-10″	0	132.8	2.7
	16	6′- 1″	1′-9″	1′-4″	11′- 1″	1′-11″	0	160.8	2.9
	18	7′- 0″	1′-9″	1′-6″	12′- 3″	2′- 0″	0	191.8	3.0
34 and 35	10	3′- 5″	1′-9″	10″	7′- 8″	1′- 8″	0	85.0	2.5
	12	4′- 3″	1′-9″	1′-0″	8′- 9″	1′- 9″	0	106.9	2.7
	14	5′- 2″	1′-9″	1′-2″	9′-11″	1′-10″	0	132.8	2.8
	16	6′- 1″	1′-9″	1′-4″	11′-1″	1′-11″	0	160.8	2.9
	18	7′- 0″	1′-9″	1′-6″	12′-4″	2′- 1″	0	192.2	3.0
36	10	3′- 5″	1′-9″	10″	7′- 8″	1′- 8″	0	85.0	2.6
	12	4′- 3″	1′-9″	1′-0″	8′-10″	1′-10″	0	107.3	2.6
	14	5′- 2″	1′-9″	1′-2″	10′- 1″	2′- 0″	0	133.3	2.7
	16	6′- 1″	1′-9″	1′-4″	11′- 3″	2′- 1″	0	161.6	2.9
	18	7′- 0″	1′-9″	1′-6″	12′- 5″	2′- 2″	0	192.6	3.0

FIG. 73B.—(*Continued.*)

38	12	4′– 0″	2′–0″	1′–0″	9′– 0″	2′– 0″	0	110.0	2.6
	14	4′–10″	2′–0″	1′–2″	10′– 2″	2′– 2″	0	135.3	2.7
	16	5′– 9″	2′–0″	1′–4″	11′– 5″	2′– 4″	0	164 2	2.8
	18	6′– 8″	2′–0″	1′–6″	12′– 8″	2′– 6″	0	195.8	2.9
	20	7′– 7″	2′–0″	1′–8″	14′– 0″	2′– 9″	0	230.3	3.0
40	12	4– 0″	2′–0″	1′–0″	9′– 0″	2′– 0″	0	110.0	2.7
	14	4′–10″	2′–0″	1′–2″	10′– 2″	2′– 2″	0	135.5	2.8
	16	5′– 9″	2′–0″	1′–4″	11′– 5″	2′– 4″	0	164.2	2.9
	18	6′– 8″	2′–0″	1′–6″	12′– 8″	2′– 6″	0	195.8	3.0
	20	7′– 7″	2′–0″	1′–8″	14′– 6″	2′– 9″	0′–6″	233 0	3.0
42	12	4′– 0″	2′–0″	1′–0″	9′– 0″	2′– 0″	0	110 0	2.8
	14	4′–10″	2′–0″	1′–2″	10′– 2″	2′– 2″	0	135.5	2.8
	16	5′– 9″	2′–0″	1′–4″	11′– 5″	2′– 4″	0	164.2	3.0
	18	6′– 8″	2′–0″	1′–6″	12′–10″	2′– 8″	0	196.5	3.0
	20	7′– 7″	2′–0″	1′–8″	14′– 6″	2′– 9″	0′– 6″	233.0	3.0

NOTES

The abutments and wing walls shown on this drawing are typical only. The Engineer will give definite elevations and dimensions for each bridge.

The depth of footings shall be determined with respect to the character of the foundation material and the possibility of undermining. All footings shall rest on a firm foundation and except where rock is encountered, shall be at a depth at least tour teet below the bed of stream or surface of ground.

The wing walls shall be designed and detailed to suit existing conditions and the angle between abutment and wing wall and the relative elevations of footings, shall be made to fit the ground.

In case it appears that the soil will not safely withstand the unit pressure noted in the above table of abutment dimensions, timber piles shall be used and spaced as shown in the table. These piles are designed for a maximum load of 15 tons. The tops of piles shall be placed below low water elevation and the estimated length of piles to be used shall be shown on the plans. In case excessive erosive action of stream is expected, piles shall be used, rip-rap, placed in front of the abutment, or other approved means taken to prevent erosion.

Concrete in abutments and wing walls shall be 1–2½–5 mix, Item 21.

Keyways between abutments and wing walls and at all horizontal joints shall comprise about 30% of the area of surface.

All exposed edges of concrete shall be chamfered one inch.

The bases of structures shown on this sheet shall be considered as approximate only, and may be ordered in writing by the Engineer, to be at any elevation and of any dimensions necessary to give a proper foundation.

Payment for furnishing and placing dowels, expansion plates and bolts will be made at the respective contract prices for Metal Reinforcement and Structural Steel (see superstructure details).

Cost of furnishing and placing material in expansion joints at ends of bridge seat will be included in the price for Item 21.

4″ vitrified clay drains on the back of abutments will be paid for under Item 7.

Porous material placed on back of wall will be paid for under Items 61, 62 and 63.

FIG. 73*B*.—(*Continued.*)

Clear span, feet	Wing walls											
	H	D	H + D	E	F	G	J	K	L	M	N	Cu. yds. 1-2½-5 concrete two wings
25 and 26	8	2'- 3"	10'- 3"	3'- 5¼"	10¼"	11¾"	1'- 4½"	1'-0¼"	14'- 3½"	12'- 4½"	7'-1¾"	43.0
	10	2'- 3"	12'- 3"	4'- 1½"	1'- 0¼"	1'-0¼"	1'- 5¼"	1'-1"	17'- 9"	15'- 4½"	8'-10½"	61.8
	12	2'- 3"	14'- 3"	4'- 9½"	1'- 2¼"	1'-1"	1'- 6"	1'-1¾"	21'- 2½"	18'- 4½"	10'- 7¼"	85.8
	14	2'- 3"	16'- 3"	5'- 6"	1'- 4¼"	1'-1½"	1'- 7"	1'-2¾"	24'- 8¼"	21'- 4½"	12'- 4"	113.6
	16	2'- 3"	18'- 3"	6'- 2½"	1'- 6¼"	1'-2"	1'- 7¾"	1'-3½"	28'- 1¾"	24'- 4½"	14'- 1"	146.8
28	8	2'- 5"	10'- 5"	3'- 6"	10½"	11¾"	1'- 4½"	1'-0¼"	14'- 7"	12'- 7½"	7'- 3½"	44.2
	10	2'- 5"	12'- 5"	4'- 2¼"	1'- 0½"	1'-0¼"	1'- 5¼"	1'-1"	18'- 0½"	15'- 7½"	9'- 0¼"	63.8
	12	2'- 5"	14'- 5"	4'-10½"	1'- 2½"	1'-1"	1'- 6"	1'-1¾"	21'- 6"	18'- 7½"	10'- 9"	87.4
	14	2'- 5"	16'- 5"	5'- 6¾"	1'- 4½"	1'-1½"	1'- 7"	1'-2¾"	24'-11½"	21'- 7½"	12'- 5¾"	116.4
	16	2'- 5"	18'- 5"	6'- 3¼"	1'- 6½"	1'-2"	1'- 7¾"	1'-3½"	28'- 5¼"	24'- 7½"	14'- 2½"	149.8
30	8	2'- 7"	10'- 7"	3'- 6½"	10½"	11¾"	1'- 4½"	1'-0¼"	14'-10½"	12'-10½"	7'- 5¼"	45.6
	10	2'- 7"	12'- 7"	4'- 2¾"	1'- 0½"	1'-0¼"	1'- 5¼"	1'-1"	18'- 4"	15'-10½"	9'- 2"	65.2
	12	2'- 7"	14'- 7"	4'-11"	1'- 2½"	1'-1"	1'- 6"	1'-1¾"	21'- 9½"	18'-10½"	10'-10¾"	89.2
	14	2'- 7"	16'- 7"	5'- 7½"	1'- 4½"	1'-1½"	1'- 7"	1'-2¾"	25'- 3"	21'-10½"	12'- 7½"	118.8
	16	2'- 7"	18'- 7"	6'- 3¾"	1'- 6½"	1'-2"	1'- 7¾"	1'-3½"	28'- 8½"	24'-10½"	14'- 4¼"	152.2
32	10	2'-11"	12'-11"	4'- 4¼"	1'- 1"	1'-0½"	1'- 6"	1'-1¾"	18'-11"	16'- 4½"	9'- 5½"	69.6
	12	2'-11"	14'-11"	5'- 0½"	1'- 3"	1'-1"	1'- 6¾"	1'-2½"	22'- 4½"	19'- 4½"	11'- 2¼"	94.6
	14	2'-11"	16'-11"	5'- 9"	1'- 5"	1'-1½"	1'- 7½"	1'-3¼"	25'-10"	22'- 4½"	12'-11"	124.2
	16	2'-11"	18'-11"	6'- 5¼"	1'- 7"	1'-2"	1'- 8¼"	1'-4"	29'- 3½"	25'- 4½"	14'- 7¾"	159.4
	18	2'-11"	20'-11"	7'- 1½"	1'- 9"	1'-2½"	1'- 9"	1'-4¾"	32'- 9"	28'- 4½"	16'- 4½"	200.8
34 and 35	10	3'- 2"	13'- 2"	4'- 5¼"	1'- 1¼"	1'-0½"	1'- 6"	1'-1¾"	19'- 4"	16'- 9"	9'- 8"	72.0
	12	3'- 2"	15'- 2"	5'- 1½"	1'- 3¼"	1'-1"	1'- 6¾"	1'-2½"	22'- 9¾"	19'- 9"	11'- 4¾"	97.8
	14	3'- 2"	17'- 2"	5'-10"	1'- 5¼"	1'-1½"	1'- 7½"	1'-3¼"	26'-3½"	22'- 9"	13'- 1¾"	128.4
	16	3'- 2"	19'- 2"	6'- 6¼"	1'- 7¼"	1'-2"	1'- 8¼"	1'-4"	29'- 8¾"	25'- 9"	14'-10½"	164.7
	18	3'- 2"	21'- 2"	7'- 2½"	1'- 9¼"	1'-2½"	1'- 9¼"	1'- 5"	33'- 2½"	28'- 9"	16'- 7¼"	207.2

FIG. 73B.—(*Continued.*)

36	10	3'- 5"	13'- 5"	4'- 6½"	1'- 1½"	1'-0½"	1'- 6"	1'-1¾"	19'- 9¼"	17'- 1½"	9'-11"	75.2
	12	3'- 5"	15'- 5"	5'- 2¾"	1'- 3½"	1'-1¼"	1'- 7"	1'-2¾"	23'- 3"	20'- 1½"	11'- 7½"	101.8
	14	3'- 5"	17'- 5"	5'-11"	1'- 5½"	1'-1¾"	1'- 8"	1'-3¾"	26'- 8½"	23'- 1½"	13'- 4¼"	133.6
	16	3'- 5"	19'- 5"	6'- 7½"	1'- 7½"	1'-2¼"	1'- 9"	1'-4¾"	30'- 2"	26'- 1½"	15'- 1"	170.6
	18	3'- 5"	21'- 5"	7'- 3¾"	1'- 9½"	1'-2¾"	1'- 9¾"	1'-5½"	33'- 7½"	29'- 1½"	16'- 9¾"	213.2
38	12	3'- 9"	15'- 9"	5'- 4"	1'- 3¾"	1'-1¼"	1'- 7¾"	1'-3½"	23'- 9¾"	20'- 7½"	11'-10¾"	107.0
	14	3'- 9"	17'- 9"	6'- 0½"	1'- 5¾"	1'-1¾"	1'- 8¾"	1'-4½"	27'- 3½"	23'- 7½"	13'- 7¾"	139.6
	16	3'- 9"	19'- 9"	6'- 8½"	1'- 7¾"	1'-2¼"	1'-10"	1'-5¾"	30'- 9"	26'- 7½"	15'- 4½"	178.2
	18	3'- 9"	21'- 9"	7'- 5"	1'- 9¾"	1'-2¾"	1'-11"	1'-6¾"	34'- 2½"	29'- 7½"	17'- 1¼"	222.8
	20	3'- 9"	23'- 9"	8'- 1¼"	1'-11¾"	1'-3¼"	2'- 0¼"	1'-8"	37'-8"	32'- 7½"	18'-10"	274.6
40	12	4'- 1"	16'- 1"	5'- 5¼"	1'- 4"	1'-1¼"	1'- 7¾"	1'-3½"	24'- 4¾"	21'- 1½"	12'- 2¼"	112.2
	14	4'- 1"	18'- 1"	6'- 1½"	1'- 6"	1'-1¾"	1'- 8¾"	1'-4½"	27'-10½"	24'- 1½"	13'-11"	145.6
	16	4'- 1"	20'- 1"	6'-10"	1'- 8"	1'-2¼"	1'-10"	1'-5¾"	31'- 4"	27'- 1½"	15'- 8"	185.0
	18	4'- 1"	22'- 1"	7'- 6¼"	1'-10"	1'-3"	1'-11"	1'-6¾"	34'- 9½"	30'- 1½"	17'- 4¾"	230.2
	20	4'- 1"	24'- 1"	8'- 2¾"	2'- 0"	1'-3½"	2'- 0¼"	1'-8"	38'- 3"	33'- 1½"	19'- 1½"	283.2
42	12	4'- 5"	16'- 5"	5'- 6¾"	1'- 4½"	1'-1½"	1'- 7¾"	1'-3½"	24'-11½"	21'- 7½"	12'- 5¾"	117.6
	14	4'- 5"	18'- 5"	6'- 3¼"	1'- 6½"	1'-2"	1'- 9"	1'-4¾"	28'- 5½"	24'- 7½"	14'- 2½"	152.0
	16	4'- 5"	20'- 5"	6'-11½"	1'- 8½"	1'-2½"	1'-10"	1'-5¾"	31'-10¾"	27'- 7½"	15'-11¼"	192.4
	18	4'- 5"	22'- 5"	7'- 7¾"	1'-10½"	1'-3"	1'-11½"	1'-7¼"	35'- 4¼"	30'- 7½"	17'- 8"	239.6
	20	4'- 5"	24'- 5"	8'- 4¼"	2'- 0½"	1'-3½"	2'- 0¼"	1'-8"	38'-10"	33'- 7½"	19'- 5"	292.6

FIG. 73B.—(*Continued.*)

Clear height H	Span of bridge	Width of base W	A	B	C	D	E	Longitudinal spacing: Row 1	Row 2	Row 3	Row 4	Row 5	
8	26′–30′	6′–8″	4′–0″	.....	.....	.....	1′– 8″	2′–6″	2′–6″				
10	26′–30′	7′– 9″	4′–0″	.....	.....	.....	2′– 9″	3′–0″	3′–0″				
	32′–36′	7′– 8″	4′–0″	.....	.....	.....	2′– 8″	2′–6″	2′–6″				
12	26′–30′	8′–10″	4′–6″	.....	.....	.....	3′– 4″	2′–6″	2′–6′				
	32′–34′	8′– 9″	2′–6″	2′–6″	.....	.....	2′– 9″	3′–0″	3′–0″	3′–0″			
	36′	8′–10″	2′–6″	2′–6″	...	.....	2′–10″	3′–0″	3′–0″	3′–0″			
	38′–42′	9′– 0″	2′–6″	2′–6″	... .	.....	3′– 0″	3′–0″	3′–0″	3′–0″			
14	26′–34′	9′–11″	2′–6″	3′–6″	... .	.....	2′–11″	2′–6″	2′–6″	2′–6″			
	36′	10′– 1″	2′–6″	3′–6″	...	.....	3′– 1″	2′–6″	2′–6″	2′–6″			
	38′–42′	10′– 2″	2′–6″	3′–6″	... .	.....	3′– 2″	2′–6″	2′–6″	2′–6″			
16	26′–30′	10′–11″	2′–6″	5′–0″	... .	.....	2′–5″	2′–6″	2′–6′	2′–6″			
	32′–34′	11′– 1″	2′–6″	5′–0″	.....	... .	2′– 7″	2′–6″	2′–6″	2′–6″			
	36′	11′– 3″	2′–6″	5′–0″	.....	.....	2′– 9″	2′–6″	2′–6″	2′–6″			
	38′–42′	11′– 5″	2′–6″	5′–0″	.....	.....	2′–11″	2′–6″	2′–6″	2′–6″			
18	32′	12′– 3″	2′–6″	2′–6″	3′–6″	.....	2′– 9″	2′–6″	2′–6″	2′–6″	2′–6″		
	34′	12′– 4″	2′–6″	2′–6″	3′–6″	.....	2′–10″	2′–6″	2′–6″	2′–6″	2′–6″		
	36′	12′– 5″	2′–6″	2′–6″	3′–6″	.....	2′–11″	2′–6″	2′–6″	2′–6″	2′–6″		
	38′–40′	12′– 8″	2′–6″	3′–0″	4′–0″	.....	2′– 2″	2′–6″	2′–6″	2′–6″	2′–6″		
	42′	12′–10″	2′–6″	3′–0″	4′–0″	.....	2′– 4″	2′–6″	2′–6″	2′–6″	2′–6″		
20	38′	14′– 0″	2′–6″	3′–5″	4′–0″	.....	3′–0″	2′–6″	2′–6″	2′–6″	2′–6″		
	40′–42′	14′– 6″	2′–6″	2′–6″	2′–6″	2′–6″	3′–6″	2′–6″	2′–6″	2′–6″	2′–6″	2′–6″	

Estimated length of piles = 20′–0″.

FIG. 73*B*.—(*Continued.*)

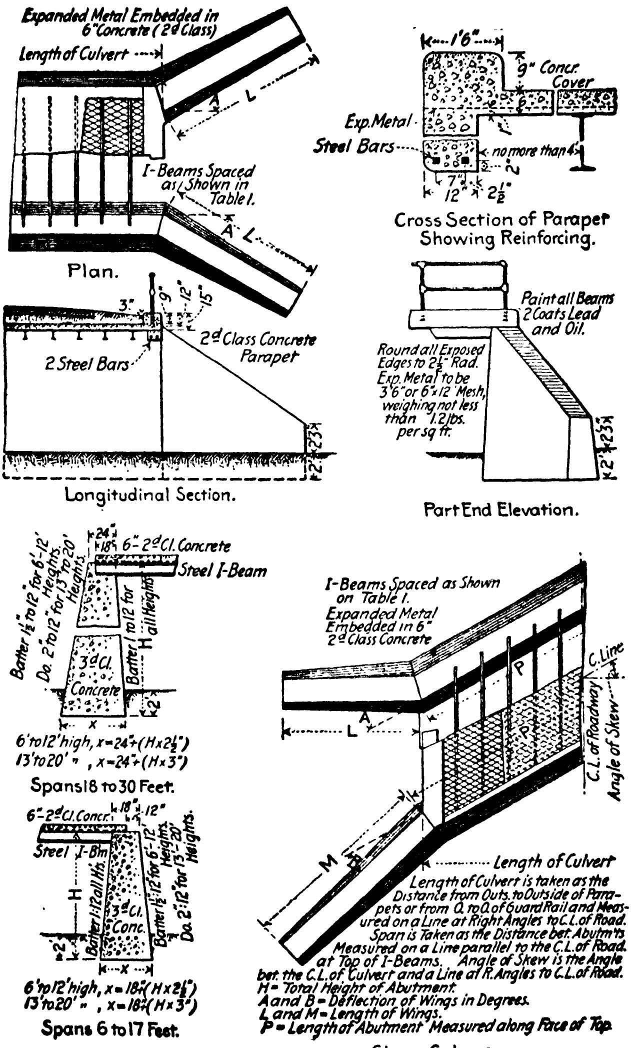


FIG. 74.—New York State I-beam bridges (1912). **H-12½ loading.** (Dimensions pages 266–269.)

Span, all Culverts	Opening at Right Angles to Faces of Abutments			Steel I-Beams					Steel Bars		Sq. feet E'x'p'dMet.		Cu. Yds. 2d Class Concrete Cover and Parapets		Ft. B. M. Lumber		Pounds Spikes and Nails	Lin. ft. Bridge Rail	Lin. ft. Pipe Rail
	15° Skew	30° Skew	45° Skew	Depth Inches	Spacing	Length feet	Wt. per ft.	Wt. of Beam	Size	Weight of 4 Bars	25 foot Length	Per foot Length	25 foot Length	Per foot Length	25 foot Length	Per foot Length			
6	5.80	5.20	4.24	6		8	12½	100	¾"	61	200	8	4.54	0.15	855	30	24	20	16
7	6.76	6.06	4.95	7		9	15	135	¾"	69	225	9	5.08	0.17	952	33	26	22	18
8	7.73	6.93	5.66	7		10	15	150	¾"	76	250	10	5.63	0.19	1042	36	28	24	20
9	8.69	7.79	6.36	8	2'-9"	11	18	198	¾"	84	275	11	6.63	0.20	1143	39	30	26	22
10	9.66	8.66	7.07	9	3'-0"	12	21	252	¾"	92	300	12	7.28	0.22	1234	42	32	28	24
11	10.63	9.53	7.78	9	3'-0"	13	21	273	⅞"	135	325	13	7.88	0.24	1325	45	34	30	26
12	11.59	10.39	8.49	9	2'-6"	14	21	294	⅞"	146	350	14	8.48	0.26	1431	48	36	32	28
13	12.56	11.26	9.19	10	3'-0"	15	25	375	1"	204	375	15	9.16	0.28	1523	51	38	34	30
14	13.52	12.12	9.90	10	2'-9"	16	25	400	1"	218	400	16	9.78	0.30	1633	54	40	36	32
15	14.49	12.99	10.61	12	3'-0"	17	31½	535½	1"	231	425	17	10.56	0.31	1727	57	42	38	34
16	15.45	13.86	11.31	12	3'-0"	18	31½	567	1"	245	450	18	11.19	0.33	1820	60	44	40	36
17	16.42	14.72	12.02	12	3'-0"	19	31½	598½	1⅛"	327	475	19	11.81	0.35	1956	63	46	42	38
18	17.39	15.59	12.73	12	3'-0"	21	31½	661½	1⅛"	361	525	21	12.90	0.39	2052	66	48	44	40
19	18.35	16.46	13.44	12	2'-9"	22	31½	693	1⅛"	378	550	22	13.51	0.41	2148	69	50	46	42
20	19.32	17.32	14.14	15	3'-0"	23	42	966	1⅛"	396	575	23	14.51	0.43	2244	72	52	48	44
21	20.28	18.19	14.85	15	3'-0"	24	42	1008	1¼"	510	600	24	15.14	0.44	2340	75	54	50	46
22	21.26	19.05	15.56	15	3'-0"	25	42	1050	1¼"	531	625	25	15.79	0.46	2519	78	56	52	48
23	22.22	19.92	16.26	15	3'-0"	26	42	1092	1¼"	552	650	26	16.43	0.48	2618	81	58	54	50
24	23.18	20.78	16.97	15	3'-0"	27	42	1134	1⅜"	694	675	27	17.08	0.50	2718	84	60	56	52
25	24.15	21.65	17.68	15	2'-9"	28	42	1176	1⅜"	720	700	28	17.72	0.52	2818	87	62	58	54
26	25.11	22.52	18.38	15	2'-6"	29	42	1218	1½"	887	725	29	18.36	0.54	2918	90	64	60	56
27	26.08	23.38	19.09	18	3'-0"	30	55	1650	1½"	918	750	30	19.50	0.56	3018	93	66	62	58
28	27.05	24.25	19.80	18	3'-0"	31	55	1705	1½"	949	775	31	20.16	0.57	3222	96	68	64	60
29	28.01	25.11	20.51	18	3'-0"	32	55	1760	1½"	979	800	32	20.82	0.59	3326	99	70	66	62
30	28.98	25.98	21.23	18	3'-0"	33	55	1815	1½"	1010	825	33	21.48	0.61	3430	102	72	68	64

NOTE: Length of Bars in Parapets same as lengths of I-Beams.

FIG. 74.—(*Continued.*)

Table No. 2			Straight A = 30° B = 30°						*Table No. 3*		15° Skew A = 30° B = 15°					
Height of Abutment	Lengths of Wings		Cubic Yards Third Class Concrete		Cubic Yards Third Class Masonry		Cubic Yds. each ft. in length of Culvert more or less than 25 ft.		Lengths of Wings		Cubic Yards Third Class Concrete		Cubic Yards Third Class Masonry		Cubic Yds. each ft. in length of Culvert more or less than 25 ft.	
H	L	M	2 Abut's	4 Wings	2 Abut's	4 Wings	Concrete	Masonry	L	M	2 Abut's	4 Wings	2 Abut's	4 Wings	Concrete	Masonry
6	3.87	3.87	23.3	5.5	28.5	6.5	0.94	1.17	3.51	3.67	24.2	5.2	29.7	6.1	0.98	1.21
7	5.60	5.60	28.3	9.2	34.3	10.9	1.17	1.41	5.06	5.40	29.6	8.8	36.0	10.4	1.20	1.47
8	7.33	7.33	34.0	13.8	40.5	16.4	1.38	1.68	6.62	7.13	35.4	12.9	42.7	15.5	1.44	1.74
9	9.06	9.06	40.0	19.4	47.3	22.7	1.63	1.95	8.16	8.86	41.6	13.3	49.6	21.6	1.69	2.03
10	10.79	10.79	46.4	25.4	54.5	30.2	1.88	2.25	9.72	10.59	48.2	24.3	57.2	28.8	1.95	2.33
11	12.52	12.52	53.3	33.4	62.3	39.1	2.16	2.56	11.27	12.32	55.2	31.7	65.1	37.1	2.24	2.66
12	14.26	14.26	60.8	42.1	70.5	49.1	2.44	2.89	12.82	14.06	62.5	39.9	73.5	46.6	2.54	3.00
13	15.99	15.99	72.2	55.5	84.7	64.3	3.01	3.49	14.38	15.79	76.8	53.0	88.4	61.3	3.12	3.62
14	17.72	17.72	81.5	68.4	94.2	73.5	3.37	3.89	15.53	17.52	86.0	64.5	98.6	74.4	3.50	4.04
15	19.45	19.45	91.0	82.4	104.2	94.0	3.75	4.31	17.48	19.25	95.7	73.4	109.3	89.6	3.89	4.47
16	21.18	21.18	101.0	98.2	114.5	111.4	4.15	4.74	19.11	20.98	106.0	93.7	120.5	106.3	4.31	4.92
17	22.92	22.92	111.1	115.9	125.3	130.7	4.56	5.19	20.58	22.72	116.1	110.1	131.8	124.5	4.74	5.39
18	24.65	24.65	121.8	134.4	136.3	152.6	5.00	5.67	22.14	24.45	127.1	128.8	143.7	145.0	5.19	5.88
19	26.38	26.38	132.7	156.2	147.7	175.7	5.45	6.16	23.69	26.18	138.3	149.0	155.8	167.3	5.66	6.38
20	28.11	28.11	144.0	179.5	159.5	201.4	5.93	6.67	25.24	27.91	150.2	170.8	168.4	191.6	6.15	6.91

FIG. 74.—*(Continued.)*

Height of Abutment	*Table No. 4* Lengths of Wings		30° Skew A = 30° B = 15° Cubic Yards Third Class Concrete		Cubic Yards Third Class Masonry		Cubic Yds. each ft. in length of Culverts more or less than 25 ft.		*Table No. 5* Lengths of Wings		45° Skew A = 45° B = 0° Cubic Yards Third Class Concrete		Cubic Yards Third Class Masonry		Cubic Yds. each ft. in length of Culvert more or less than 25 ft.	
H	L	M	2 Abut's	4 Wings	2 Abut's	4 Wings	Concrete	Masonry	L	M	2 Abut's	4 Wings	2 Abut's	4 Wings	Concrete	Masonry
6	3.4	4.44	27.0	5.6	33.3	6.8	1.09	1.35	3.6	4.24	33.0	5.5	40.5	6.8	1.34	1.65
7	4.9	6.56	32.8	8.8	40.2	11.3	1.33	1.63	5.1	6.36	40.5	9.7	49.5	11.2	1.63	2.00
8	6.4	8.69	39.5	14.4	47.6	17.0	1.59	1.94	6.6	8.49	48.3	14.5	58.8	17.1	1.95	2.37
9	7.9	10.81	46.3	20.2	55.7	23.8	1.87	2.26	8.1	10.81	57.0	20.3	68.5	23.8	2.29	2.76
10	9.4	12.93	53.7	26.9	64.0	31.7	2.17	2.60	9.6	12.73	66.0	27.2	78.5	31.8	2.66	3.18
11	10.9	15.05	61.6	34.7	72.9	40.9	2.48	2.95	11.1	14.85	75.4	35.0	89.2	41.0	3.04	3.63
12	12.4	17.17	70.0	43.7	82.3	51.4	2.82	3.34	12.6	16.97	85.5	44.1	100.5	51.4	3.46	4.09
13	13.9	19.30	85.7	58.3	99.2	67.5	3.48	4.03	14.1	19.10	105.0	53.8	121.5	67.9	4.26	4.94
14	15.4	21.42	96.0	71.4	110.7	82.8	3.89	4.49	15.6	21.22	117.6	71.8	135.3	83.0	4.76	5.50
15	16.9	23.54	107.0	86.4	122.4	99.5	4.33	4.97	17.1	23.54	130.8	87.1	149.8	99.4	5.31	6.09
16	18.4	25.66	118.0	102.9	134.6	117.3	4.79	5.48	19.6	25.46	144.7	103.6	165.0	117.4	5.87	6.71
17	19.9	27.78	129.7	121.2	147.0	137.2	5.27	6.00	20.1	27.58	159.0	121.6	180.6	137.8	6.46	7.35
18	21.4	29.90	142.0	141.1	160.5	159.7	5.77	6.54	21.6	29.70	173.0	141.8	197.0	159.7	7.08	8.02
19	22.9	32.02	154.6	163.5	174.0	184.4	6.30	7.11	23.1	31.82	189.6	164.1	213.9	184.7	7.72	8.71
20	24.4	34.14	168.0	188.5	188.0	211.2	6.84	7.70	24.6	33.94	206.0	189.1	232.0	211.6	8.39	9.43

FIG. 74.—(*Continued.*)

Table No. 6	Number I-Beams For Concrete Covers only			P = Length of Abutments		
	Spacing					
Length of Culvert	2'-6"	2'-9"	3'-0"	15° Skew	30° Skew	45° Skew
18	5	5	4	18.64	20.79	25.46
19	5	5	5	19.67	21.94	26.97
20	6	5	5	20.71	23.09	28.28
21	6	6	5	21.74	24.25	29.70
22	6	6	5	22.78	25.40	31.11
23	7	6	6	23.81	26.66	32.53
24	7	6	6	24.85	27.71	33.94
25	7	7	6	25.88	28.87	35.36
26	8	8	7	26.92	30.02	36.77
27	8	8	7	27.95	31.18	38.18
28	9	8	7	28.99	32.33	39.60
29	9	8	8	30.02	33.49	41.01
30	9	9	8	31.06	34.64	42.43
31	10	9	9	32.09	35.80	43.84
32	10	9	9	33.13	36.95	45.26
33	11	10	9	34.16	38.10	46.67

Application of Tables

Quantities for a 30° Skew Concrete Culvert, concrete top, length 30 feet, opening 13 feet high and 12 feet wide. From Table 1, an opening 12.12 ft. wide 30° Skew is a 14-ft. span requiring (see 30-ft. length, Table 6) 9 I-Beams spaced 2'-9" c. to c. (9 × 400) = 3600 lbs. I-Beams; 218 lbs. Bars; 400 + (5 × 16) = 480 sq. ft. Ex'p'd Metal; 9.78 + (5 × 30) = 11.28 cu. yds. 2d class Concrete 32 lin. ft. Pipe Rail. An opening 13 ft. high will require Abutments, 16 ft. high (13' + 2' in ground + 10" I-Beam = 15'-10"). From Table 4, Abutments = 118.0 cu. yds., Wings = 102.9 cu. yds. (5 × 4.79 = 23.95 cu. yds. 5 ft. extra length of Culvert) 118.0 + 102.9 + 23.95 = 244.85 cu. yds. 3d Class Concrete.

For Spans of more than 17 feet, use Masonry Tables for Concrete Abutments and Wings.

FIG. 74.—(*Continued.*)

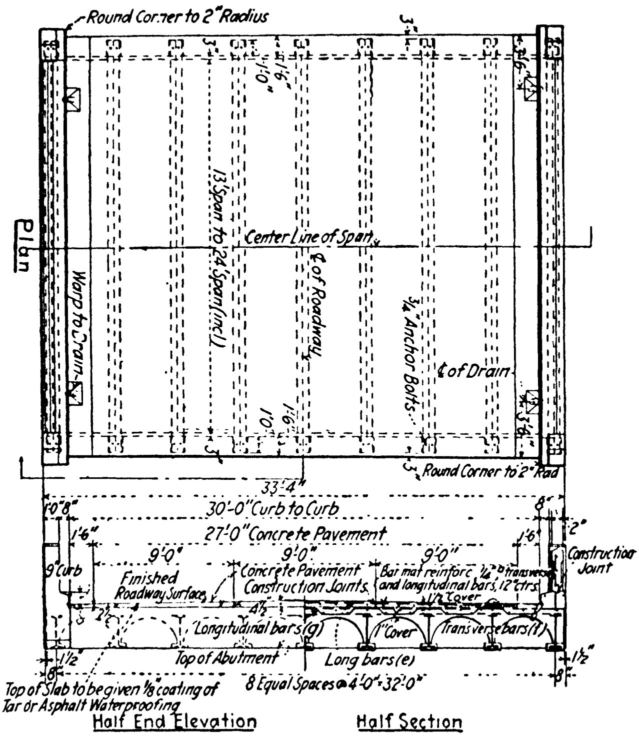


FIG. 75.—Steel I-beam stringer superstructures. (H-20 loading.) New York 1926. Spans 13 to 25 feet.

Detail dimensions pages 272–274. (Abutments for spans 13 to 25 feet, see page 241.)

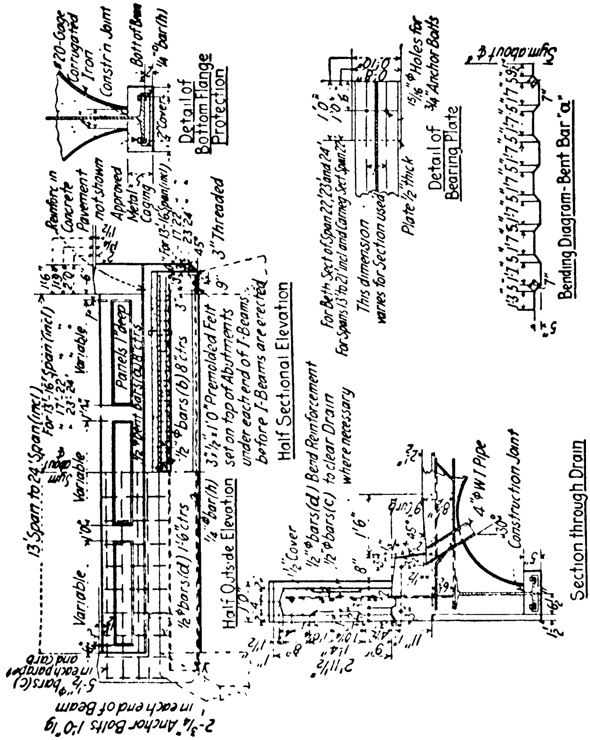


FIG. 75.—(*Continued.*)

Clear span, ft.	Section modulus required	I-beams				Summary of quantities required (no overrun or rounding)						
		Size	Type	No. req.	Length	Cu. yds. concrete		Bbls. cement	Lbs. metal reinforcement	Sq. ft. waterproofing	Sq. ft. corrugated iron	Lbs. structural steel
						1-1½-3	1-2-3½					
13	61.8	15″ -50#	Carnegie	9	15′-6″	7.0	25.4	57¼	2,120	439	460	7,290
	61.8	15 5/32″-42.5#	Beth.	9	15′-6″	7.0	25.4	57¼	2,120	439	460	6,240
14	68.3	18″ -48.2#	Carnegie	9	16′-6″	7.4	29.2	64½	2,290	467	540	7,470
	68.3	18″ -49#	Beth.	9	16′-6″	7.4	29.2	64½	2,290	467	540	7,590
15	74.2	18″ -48.2#	Carnegie	9	17′-6″	7.9	30.7	68	2,420	494	570	7,910
	74.2	18″ -49#	Beth.	9	17′-6″	7.9	30.7	68	2,420	494	570	8,030
16	80.2	18″ -48.2#	Carnegie	9	18′-6″	8.2	32.3	71½	2,580	521	610	8,340
	80.2	18″ -49#	Beth.	9	18′-6″	8.2	32.3	71½	2,580	521	610	8,470
17	86.6	18″ -54.7#	Carnegie	9	19′-6″	8.7	33.7	75	2,730	549	650	9,910
	86.6	18″ -49#	Beth.	9	19′-6″	8.7	34.0	75½	2,730	549	650	8,910
18	93.2	18″ -60#	Carnegie	9	20′-6″	9.2	35.2	78½	2,890	576	680	11,380
	93 2	18 1/16″-52	Beth.	9	20′-6″	9.2	35.5	79	2,890	576	680	9,910
19	101.2	20″ -65.4#	Carnegie	9	21′-6″	9.6	38.4	84½	3,030	604	750	12,970
	101.2	20″ -59.5#	Beth.	9	21′-6″	9.6	38.8	85¼	3,030	604	750	11,830
20	108.1	20″ -65.4#	Carnegie	9	22′-6″	10.0	39.8	87¾	3,190	631	790	13,560
	108.2	20″ -59.5#	Beth.	9	22′-6″	10.0	40.3	88½	3,190	631	790	12,360
21	115.2	20″ -65.4#	Carnegie	9	23′-6″	10.4	41.6	91½	3,320	659	830	14,150
	115.3	20″ -59.5#	Beth.	9	23′-6″	10.4	41.9	92¼	3,320	659	830	12,900
22	123.2	20″ -75#	Carnegie	9	24′-6″	10.9	43.2	95½	3,490	686	870	16,850
	123.2	20⅛″ -64.5#	Beth	9	24′-6″	10.9	43.8	96½	3,490	686	870	14,600
23	133.7	24″ -74.2#	Carnegie	9	25′-6″	11.4	48.4	105¼	3,630	713	1,020	17,400
	130.0	18⅛″ -74#	Beth.	9	25′-6″	11.4	43.6	97	3,630	713	870	17,360
24	141.1	24″ -74.2#	Carnegie	9	26′-6″	11.8	50.2	109	3,780	741	1,060	18,070
	139.0	20″ -73#	Beth.	9	26′-6″	11.8	47.2	104	3,780	741	940	17,790

FIG. 75.—(*Continued.*)

NOTES

The above bar list makes no provision for splicing reinforcing bars except longitudinal ¼″ □ bars "e and h." Bars may be spliced at places approved by the Engineer. Bars so spliced shall be lapped 40 diameters and spaced to allow 1″ of concrete between them.

All concrete except in concrete pavement and parapet shall be Nominal Mix 1 : 2 : 3½″–2200#/□″.

Concrete in wearing surface and parapet to be Nominal Mix 1 : 1½ : 3–2500#/□″.

The concrete used for encasement of bottom flanges of I's shall be made of stone not to exceed ¾ inch in size.

No construction joints other than those shown on the plans will be permitted without written permission of the Engineer. Construction joints to be free from laitance.

All reinforcing bars and structural steel shall be of medium open hearth steel. All dimensions for bending reinforcing bars are center to center of bars.

All parapet, fascia and curb surfaces shall be given a rubbed finish, cost to be included in the unit price for 1 : 1½ : 3 concrete.

The cost of furnishing and installing pipe drains, metal caging and premoulded felt strips shall be included in the unit price for 1 : 2 : 3½ concrete.

FIG. 75.—(*Continued.*)

Bar list									
Clear span, ft.	Mark	No.	Size	Length	Clear span, ft.	Mark	No.	Size	Length
13	*a*	21	½″ φ	35′- 7″	14	*a*	22	½″ φ	35′- 7″
	b	40	½″ φ	32′-11″		*b*	44	½″ φ	32′-11″
	c	10	½″ φ	15′- 8″		*c*	10	½″ φ	16′- 8″
	d	24	½″ φ	4′- 2″		*d*	26	½″ φ	4′- 2″
	e	27	¼″ □	15′- 6″		*e*	54	¼″ □	8′- 8″
	f	48	¼″ □	8′- 8″		*f*	51	¼″ □	8′- 8″
	g	33	½″ φ	15′- 6″		*g*	33	½″ φ	16′- 6″
	h	18	¼″ □	12′- 6″		*h*	18	¼″ □	13′- 6″
15	*a*	24	½″ φ	35′- 7″	16	*a*	25	½″ φ	35′- 7″
	b	46	½″ φ	32′-11″		*b*	50	½″ φ	32′-11″
	c	10	½″ φ	17′- 8″		*c*	10	½″ φ	18′- 8″
	d	26	½″ φ	4′- 2″		*d*	28	½″ φ	4′- 2″
	e	54	¼″ □	9′- 2″		*e*	54	¼″ □	9′- 8″
	f	54	¼″ □	8′- 8″		*f*	57	¼″ □	8′- 8″
	g	33	½″ φ	17′- 6″		*g*	33	½″ φ	18′- 6″
	h	18	¼″ □	14′- 6″		*h*	18	¼″ □	15′- 6″
17	*a*	27	½″ φ	35′- 7″	18	*a*	28	½″ φ	35′- 7″
	b	52	½″ φ	32′-11″		*b*	56	½″ φ	32′-11″
	c	10	½″ φ	20′- 2″		*c*	10	½″ φ	21′- 2″
	d	30	½″ φ	4′- 2″		*d*	30	½″ φ	4′- 2″
	e	54	¼″ □	10′- 2″		*e*	54	¼″ □	10′- 8″
	f	60	¼″ □	8′- 8″		*f*	63	¼″ □	8′- 8″
	g	33	½″ φ	19′- 6″		*g*	33	½″ φ	20′- 6″
	h	36	¼″ □	8′- 8″		*h*	36	¼″ □	9′- 2″
19	*a*	30	½″ φ	35′- 7″	20	*a*	31	½″ φ	35′- 7″
	b	58	½″ φ	32′-11″		*b*	62	½″ φ	32′-11″
	c	10	½″ φ	22′- 2″		*c*	10	½″ φ	23′- 2″
	d	32	½″ φ	4′- 2″		*d*	34	½″ φ	4′- 2″
	e	54	¼″ □	11′- 2″		*e*	54	¼″ □	11′- 8″
	f	66	¼″ □	8′- 8″		*f*	69	¼″ □	8′- 8″
	g	33	½″ φ	21′- 6″		*g*	33	½″ φ	22′- 6″
	h	36	¼″ □	9′- 8″		*h*	36	¼″ □	10′- 2″
21	*a*	33	½″ φ	35′- 7″	22	*a*	34	½″ φ	35′- 7″
	b	64	½″ φ	32′-11″		*b*	68	½″ φ	32′-11″
	c	10	½″ φ	24′- 2″		*c*	10	½″ φ	25′- 2″
	d	34	½″ φ	4′- 2″		*d*	36	½″ φ	4′- 2″
	e	54	¼″ □	12′- 2″		*e*	54	¼″ □	12′- 8″
	f	72	¼″ □	8′- 8″		*f*	75	¼″ □	8′- 8″
	g	33	½″ φ	23′- 6″		*g*	33	½″ φ	24′- 6″
	h	36	¼″ □	10′- 8″		*h*	36	¼″ □	11′- 2″
23	*a*	36	½″ φ	35′- 7″	24	*a*	37	½″ φ	35′- 7″
	b	70	½″ φ	32′-11″		*b*	74	½″ φ	32′-11″
	c	10	½″ φ	26′- 8″		*c*	10	½″ φ	27′- 8″
	d	38	½″ φ	4′- 2″		*d*	38	½″ φ	4′- 2″
	e	54	¼″ □	13′- 2″		*e*	54	¼″ □	13′- 8″
	f	78	¼″ □	8′- 8″		*f*	81	¼″ □	8′- 8″
	g	33	½″ φ	25′- 6″		*g*	33	½″ φ	26′- 6″
	h	36	¼″ □	11′- 8″		*h*	36	¼″ □	12′- 2″

a. Bent transverse bars—slab.
b. Straight transverse bars—slab.
c. Straight longitudinal bars in parapet and curb
d. Straight vertical bars in parapet and curb.
e. Straight longitudinal bars bar mat reinforcement.
f. Straight transverse bars bar mat reinforcement.
g. Straight longitudinal bars slab—top and bottom.
h. Straight longitudinal bars under I-beams.

FIG. 75.—(*Continued.*)

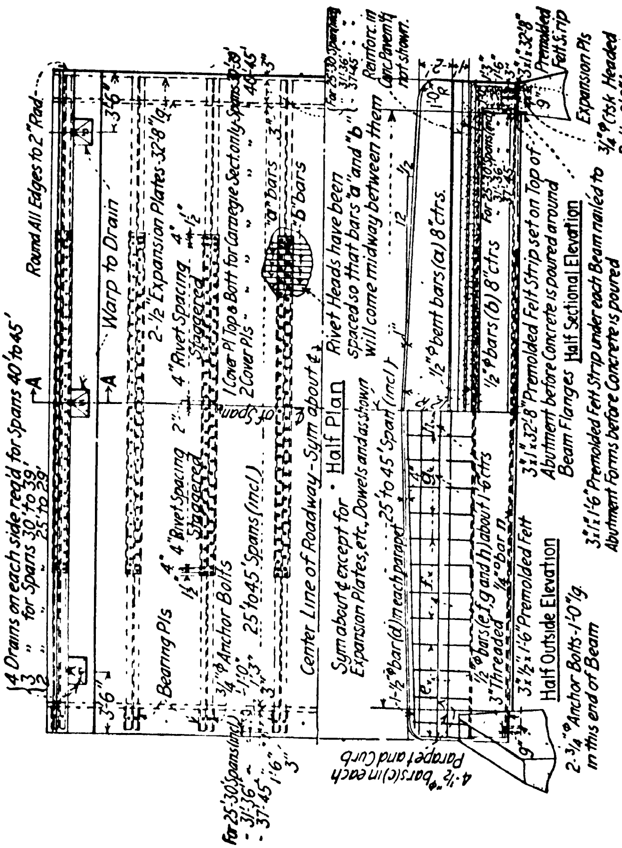


Fig. 76A.—Steel I beam superstructures (H-20 loading). Spans 25 to 45 feet. New York State Standards 1926. (Dimensions pages 278–283.)

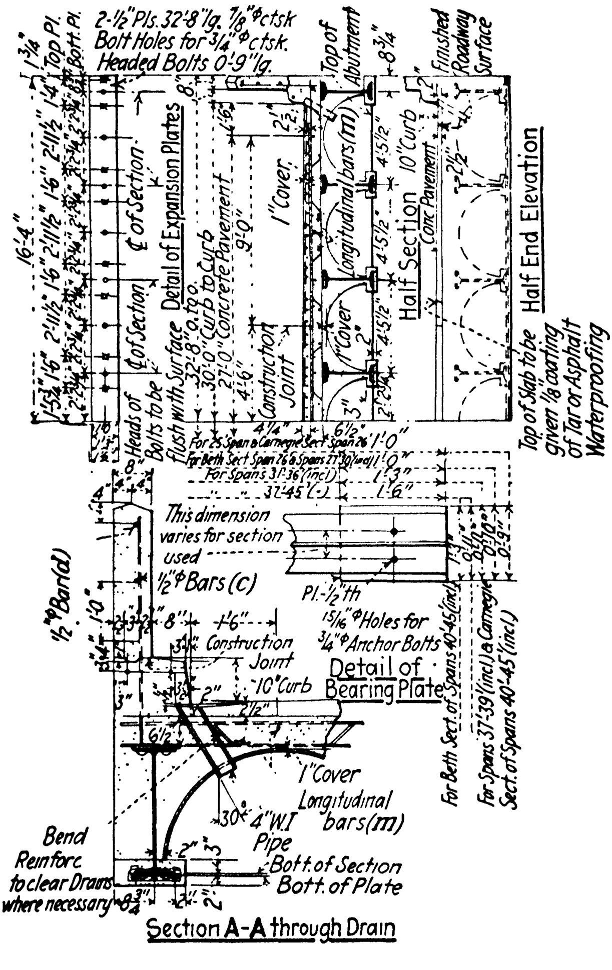


FIG. 76A.—(Continued.)

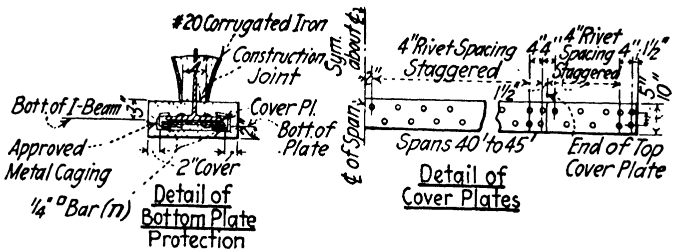


FIG. 76A.—(*Continued.*)

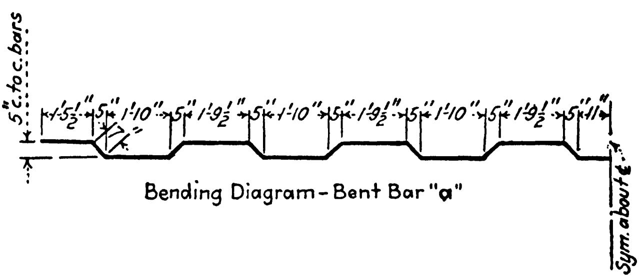


FIG. 76A (*Cont.*).—Bar bending diagram. Floor slab. I-beam stringer bridge. 30 ft. roadway.

Clear span, ft.	I-beams					Summary of quantities required (no overrun or rounding)							
						Item No. 17	Item No. 51-D	Item No. 16	Item No. 15	Item No. 60-A	Item No. 30	Item No. 32-B	Item No. 32-A
	Section modulus required	Size	Type	No. required	Length	Cu. yds. concrete parapet	Cu. yds. concrete pavement	Cu. yds. concrete for structures	Bbls. Portland cement	Sq. ft. waterproofing	Sq. yds. reinforcement for pavement	Lbs. structural steel	Lbs. reinforcement for structures
25	172.0	24″–79.9#	Carnegie	8	27′– 6″	3.2	7.9	51.0	108¾	768	86.6	19,130	3,440
	172.0	24″–73.5#	Beth.	8	27′– 6″	3.2	7.9	51.4	109½	768	86.6	17,720	3,440
26	182.5	24″–90#	Carnegie	8	28′– 6″	3.3	8.2	52.7	112½	795	89.5	22,070	3,590
	182.5	24 3/32″–79.5#	Beth.	8	28′– 6″	3.3	8.2	53.5	113¾	795	89.5	19,690	3,590
27	195.0	27″–90#	Carnegie	8	29′– 6″	3.4	8.4	57.0	120½	823	92.5	22,800	3,710
	194.0	24″–84.5#	Beth.	8	29′– 6″	3.4	8.4	55.0	117	823	92.5	21,510	3,710
28	206.2	27″–90#	Carnegie	8	30′– 6″	3.5	8.7	58.7	124	850	95.4	23,530	3,850
	205.0	25⅞″–85.5#	Beth.	8	30′– 6″	3.5	8.7	58.1	123	850	95.4	22,430	3,850
29	217.0	27″–90#	Carnegie	8	31′– 6″	3.7	9.0	60.4	127¾	877	98.2	24,240	3,970
	216.2	26″–91#	Beth.	8	31′– 6″	3.7	9.0	60.2	127¼	877	98.2	24,500	3,970
30	233.0	27″–90# 2–10″ × ⅜ Pls.	Carnegie	8	32′– 6″ 9′–11″	3.8	9.3	63.1	133	905	101.1	27,120	4,120
	229.0	24″–99.5#	Beth.	8	32′– 6″	3.8	9.3	60.2	128	905	101.1	27,430	4,120

FIG. 76A.—*(Continued.)*

31	248.0	27″–90# 2–10 × 3/8″ Pls.	Carnegie	8	34′– 0″ 13′– 3″	4.0	9.8	66.7	140¾	946	105.4	29,310	4,250
	243.5	26⅛″–98#	Beth.	8	34′– 0″	4.0	9.8	65.3	138¼	946	105.4	28,610	4,250
32	260.0	27″–90# 2–10″ × 3/8 Pls.	Carnegie	8	35′– 0″ 15′–3″	4.1	10.0	68.4	144¼	973	108.3	30,470	4,400
	258.0	27⅞″–100#	Beth.	8	35′– 0″	4.1	10.0	68.5	144¼	973	108.3	29,950	4,400
33	272.5	27″–90# 2–10″ × 3/8 Pls.	Carnegie	8	36′– 0″ 17′– 3″	4.3	10.3	70.2	148¼	1,001	111.1	31,620	4,520
	272.0	28″–106#	Beth.	8	36′– 0″	4.3	10.3	70.5	148½	1,001	111.1	32,480	4,520
34	285.5	27″–90# 2–10″ × 3/8 Pls.	Carnegie	8	37′– 0″ 19′– 3″	4.4	10.6	72.0	152	1,028	114.1	32,770	4,670
	284.0	28″–106#	Beth.	8	37′– 0″	4.4	10.6	72.2	152¼	1,028	114.1	33,330	4,670
35	298.0	27″–90# 2–10″ × 7/16 Pls.	Carnegie	8	38′– 0″ 21′– 3″	4.6	10.0	73.7	155½	1,056	117.0	34,650	4,780
	298.0	28⅛″ × 113#	Beth.	8	38′– 0″	4.6	10.0	74.0	156	1,056	117.0	36,310	4,780
36	313.0	27″–90# 2–10″ × ½″ Pls.	Carnegie	8	39′– 0″ 22′– 7″	4.7	11.1	75.7	159¾	1,083	120.0	36,460	4,930
	312.0	29⅞″–115#	Beth.	8	39′– 0″	4.7	11.1	77.3	162½	1,083	120.0	36,830	4,930
37	331.0	27″–90# 2–10″ × 9/16 Pls.	Carnegie	8	40′– 6″ 23′–11″	4.9	11.5	79.3	167¼	1,124	127.7	39,120	5,070
	330.0	30″–121#	Beth.	8	40′– 6″	4.9	11.5	81.2	170½	1,124	127.7	41,540	5,070
38	344.5	27″–90# 2–10″ × 5/8 Pls.	Carnegie	8	41′– 6″ 25′–11″	5.0	11.8	81.4	171½	1,152	130.5	41,350	5,210
	344.0	30″–121#	Beth.	8	41′– 6″	5.0	11.8	83.0	174¼	1,152	130.5	42,500	5,210

FIG. 76A.—(*Continued.*)

Clear span, ft.	I-beams					Summary of quantities required (no overrun or rounding)							
						Item No. 17	Item No. 51-D	Item No. 16	Item No. 15	Item No. 60-A	Item No. 30	Item No. 32-B	Item No. 32-A
	Section modulus required	Size	Type	No. required	Length	Cu. yds. concrete parapet	Cu. yds. concrete pavement	Cu. yds. concrete for structures	Bbls. portland cement	Sq. ft. waterproofing	Sq. yds. reinforcement for pavement	Lbs. structural steel	Lbs. reinforcement for structures
39	360.0	27"–90# 2–10" × 1 1/16 Pls.	Carnegie	8	42'– 6" 27'– 3"	5.2	12.1	83.2	175 3/4	1,180	133.4	43,480	5,330
	359.0	30 1/8"–129#	Beth.	8	42'– 6"	5.2	12.1	84.9	178 1/2	1,180	133.4	46,190	5,330
40	374.0	27"–90# 2–10" × 3/8" Pls. 2–10" × 3/8 Pls.	Carnegie	8	43'– 6" 28'– 7" 25'– 3"	5.3	12.4	85.4	180	1,207	136.2	45,010	5,470
	376.0	25 7/8"–144#	Beth.	8	43'– 6"	5.3	12.4	83.9	177 1/2	1,207	136.2	52,520	5,470
41	389.0	27"–90# 2–10" × 7/16 Pls. 2–10" × 3/8" Pls.	Carnegie	8	44'–6" 29'–11" 25'– 3"	5.5	12.6	87.0	183 3/4	1,234	139.0	47,040	5,600
	393.0	26"–151#	Beth.	8	44'– 6"	5.5	12.6	86.2	182 1/4	1,234	139.0	56,170	5,600
42	406.0	27"–90# 2–10" × 7/16" Pls. 2–10" × 7/16" Pls.	Carnegie	8	45'– 6" 31'–11" 27'– 3"	5.6	13.0	89.1	188	1,261	142.0	49,600	5,740
	408.0	26"–151#	Beth.	8	45'– 6"	5.6	13.0	83.3	178 1/4	1,261	142.0	57,380	5,740

FIG. 76A.—(*Continued.*)

43	421.0	27″–90# 2–10″ × 7/16 Pls. 2–10″ × 1/2 Pls.	Carnegie	8	46′–6″ 27′–3″ 33′–3″	5.8	13.2	91.1	192 1/4	1,289	144.8	51,780	5,860
	424.0	26 1/8″–160#	Beth.	8	46′–6″	5.8	13.2	90.1	190 1/4	1,289	144.8	61,930	5,860
44	439.0	27″–90# 2–10″ × 1/2″ Pls. 2–10″ × 1/2″ Pls.	Carnegie	8	47′–6″ 33′–11″ 27′–11″	5.9	13.5	93.3	196 3/4	1,316	147.8	53,790	6,010
	444.0	28″–165#	Beth.	8	47′–6″	5.9	13.5	93.4	196 3/4	1,316	147.8	65,110	6,010
45	452.0	27″–90# 2–10″ × 9/16 Pls. 2–10″ × 1/2″ Pls.	Carnegie	8	48′–6″ 35′–11″ 27′–11″	6.1	13.8	95.1	201.0	1,344	150.7	56,290	6,120
	460.0	28″–165#	Beth.	8	48′–6″	6.1	13.8	95.2	201 1/4	1,344	150.7	66,430	6,120

NOTES

The bar list shown makes no provision for splicing reinforcing bars except longitudinal 1/4″ □ bar ("n"). Bars may be spliced at places approved by the Engineer. Bars so spliced shall be lapped 40 diameters.

Concrete for pavement shall be 1 : 1 1/2 : 3 mix, all other concrete 1 : 2 : 3 1/2 mix.

The concrete used for encasement of bottom flanges of L's shall be made of stone not to exceed 3/4″ in size.

No construction joints other than those shown on the plans will be permitted without written permission of the Engineer. Construction joints to be free from laitance.

A layer of graphite grease about 1/8″ thick shall be placed between the expansion plates.

All reinforcing bars and structural steel shall be of medium open hearth steel. All dimensions for bending reinforcing bars are center to center of bars.

All parapet, fascia and curb surfaces shall be given a carborundum rubbed finish, cost to be included in the unit price for Items No. 16 and No. 17.

The cost of furnishing and installing pipe drains, metal caging, corrugated iron and premoulded felt strips shall be included in the unit price for Item No. 16.

All rivets shall be 3/4″ϕ.

The lower expansion plate shall be placed on ton of abutment before concrete in abutment has set.

For name plate—see specifications.

Place 2-5/8″ϕ eyebolts, 1′–6″ long in each end of each parapet as directed by the Engineer for the attachment of cable guide railing. Cost of furnishing and placing to be included in the unit price for Item No. 17.

FIG. 76A.—(*Continued.*)

Bar list

Clear span, ft.	Mark	Size	No.	Length	Clear span, ft.	Mark	Size	No.	Length	Clear span, ft.	Mark	Size	No.	Length
25	*a*	½″ φ	39	34′-8″	28	*a*	½″ φ	43	34′-8″	31	*a*	½″ φ	48	34′-8″
	b	½″ φ	76	32′-4″		*b*	½″ φ	86	32′-4″		*b*	½″ φ	94	32′-4″
	c	½″ φ	8	27′-8″		*c*	½″ φ	8	30′-8″		*c*	½″ φ	8	34′-2″
	d	½″ φ	2	27′-0″		*d*	½″ φ	2	30′-0″		*d*	½″ φ	2	33′-6″
	e	½″ φ	20	3′-3″		*e*	½″ φ	20	3′-3″		*e*	½″ φ	20	3′-3″
	f	½″ φ	16	3′-6″		*f*	½″ φ	16	3′-6″		*f*	½″ φ	16	3′-6″
	g	½″ φ	4	3′-9″		*g*	½″ φ	8	3′-9″		*g*	½″ φ	14	3′-9″
	m	½″ φ	29	27′-6″		*m*	½″ φ	29	30′-6″		*m*	½″ φ	29	34′-0″
	n	¼″ □	32	12′-8″		*n*	¼″ □	32	14′-2″		*n*	¼″ □	32	15′-8″
26	*a*	½″ φ	40	34′-8″	29	*a*	½″ φ	45	34′-8″	32	*a*	½″ φ	49	34′-8″
	b	½″ φ	80	32′-4″		*b*	½″ φ	88	32′-4″		*b*	½″ φ	98	32′-4″
	c	½″ φ	8	28′-8″		*c*	½″ φ	8	31′-8″		*c*	½″ φ	8	35′-2″
	d	½″ φ	2	28′-0″		*d*	½″ φ	2	31′-0″		*d*	½″ φ	2	34′-6″
	e	½″ φ	20	3′-3″		*e*	½″ φ	20	3′-3″		*e*	½″ φ	20	3′-3″
	f	½″ φ	16	3′-6″		*f*	½″ φ	16	3′-6″		*f*	½″ φ	16	3′-6″
	g	½″ φ	6	3′-9″		*g*	½″ φ	10	3′-9″		*g*	½″ φ	14	3′-9″
	m	½″ φ	29	28′-6″		*m*	½″ φ	29	31′-6″		*m*	½″ φ	29	35′-0″
	n	¼″ □	32	13′-2″		*n*	¼″ □	32	14′-8″		*n*	¼″ □	48	11′-1″
27	*a*	½″ φ	42	34′-8″	30	*a*	½″ φ	46	34′-8″	33	*a*	½″ φ	51	34′-8″
	b	½″ φ	82	32′-4″		*b*	½″ φ	92	32′-4″		*b*	½″ φ	100	32′-4″
	c	½″ φ	8	29′-8″		*c*	½″ φ	8	32′-8″		*c*	½″ φ	8	36′-2″
	d	½″ φ	2	29′-0″		*d*	½″ φ	2	32′-0″		*d*	½″ φ	2	35′-6″
	e	½″ φ	20	3′-3″		*e*	½″ φ	20	3′-3″		*e*	½″ φ	20	3′-3″
	f	½″ φ	16	3′-6″		*f*	½″ φ	16	3′-6″		*f*	½″ φ	16	3′-6″
	g	½″ φ	8	3′-9″		*g*	½″ φ	12	3′-9″		*g*	½″ φ	16	3′-9″
	m	½″ φ	29	29′-6″		*m*	½″ φ	29	32′-6″		*m*	½″ φ	29	36′-0″
	n	¼″ □	32	13′-8″		*n*	¼″ □	32	15′-2″		*n*	¼″ □	48	11′-5″

FIG. 76A.—(*Continued.*)

Bar list																			
Clear span, ft.	Mark	Size	No.	Length	Clear span, ft.	Mark	Size	No.	Length	Clear span, ft.	Mark	Size	No.	Length	Clear span, ft.	Mark	Size	No.	Length
	a	½″φ	52	34′-8″		a	½″φ	57	34′-8″		a	½″φ	61	34′-8″		a	½″φ	66	34′-8″
	b	½″φ	104	32′-4″		b	½″φ	112	32′-4″		b	½″φ	122	32′-4″		b	½″φ	130	32′-4″
	c	½″φ	8	37′-2″		c	½″φ	8	40′-8″		c	½″φ	8	43′-8″		c	½″φ	8	46′-8″
	d	½″φ	2	36′-6″		d	½″φ	2	40′-0″		d	½″φ	2	43′-0″		d	½″φ	2	46′-0″
34	e	½″φ	20	3′-3″	37	e	½″φ	20	3′-3″	40	e	½″φ	20	3′-3″	43	e	½″φ	20	3′-3″
	f	½″φ	16	3′-6″		f	½″φ	16	3′-6″		f	½″φ	16	3′-6″		f	½″φ	16	3′-6″
	g	½″φ	16	3′-9″		g	½″φ	16	3′-9″		g	½″φ	16	3′-9″		g	½″φ	16	3′-9″
	h	½″φ	2	4′-0″		h	½″φ	6	4′-0″		h	½″φ	10	4′-0″		h	½″φ	14	4′-0″
	m	½″φ	29	37′-0″		m	½″φ	29	40′-6″		m	½″φ	29	43′-6″		m	½″φ	29	46′-6″
	n	¼″□	48	11′-9″		n	¼″□	48	12′-9″		n	¼″□	48	13′-9″		n	¼″□	48	14′-9″
	a	½″φ	54	34′-8″		a	½″φ	58	34′-8″		a	½″φ	63	34′-8″		a	½″φ	67	34′-8″
	b	½″φ	106	32′-4″		b	½″φ	116	32′-4″		b	½″φ	124	32′-4″		b	½″φ	134	32′-4″
	c	½″φ	8	38′-2″		c	½″φ	8	41′-8″		c	½″φ	8	44′-8″		c	½″φ	8	47′-8″
	d	½″φ	2	37′-6″		d	½″φ	2	41′-0″		d	½″φ	2	44′-0″		d	½″φ	2	47′-0″
35	e	½″φ	20	3′-3″	38	e	½″φ	20	3′-3″	41	e	½″φ	20	3′-3″	44	e	½″φ	20	3′-3″
	f	½″φ	16	3′-6″		f	½″φ	16	3′-6″		f	½″φ	16	3′-6″		f	½″φ	16	3′-6″
	g	½″φ	16	3′-9″		g	½″φ	16	3′-9″		g	½″φ	16	3′-9″		g	½″φ	16	3′-9″
	h	½″φ	2	4′-0″		h	½″φ	8	4′-0″		h	½″φ	12	4′-0″		h	½″φ	16	4′-0″
	m	½″φ	29	38′-0″		m	½″φ	29	41′-6″		m	½″φ	29	44′-6″		m	½″φ	29	47′-6″
	n	¼″□	48	12′-1″		n	¼″□	48	13′-1″		n	¼″□	48	14′-1″		n	¼″□	48	15′-1″
	a	½″φ	55	34′-8″		a	½″φ	60	34′-8″		a	½″φ	64	34′-8″		a	½″φ	69	34′-8″
	b	½″φ	110	32′-4″		b	½″φ	118	32′-4″		b	½″φ	128	32′-4″		b	½″φ	136	32′-4″
	c	½″φ	8	39′-2″		c	½″φ	8	42′-8″		c	½″φ	8	45′-8″		c	½″φ	8	48′-8″
	d	½″φ	2	38′-6″		d	½″φ	2	42′-0″		d	½″φ	2	45′-0″		d	½″φ	2	48′-0″
36	e	½″φ	20	3′-3″	39	e	½″φ	20	3′-3″	42	e	½″φ	20	3′-3″	45	e	½″φ	20	3′-3″
	f	½″φ	16	3′-6″		f	½″φ	16	3′-6″		f	½″φ	16	3′-6″		f	½″φ	16	3′-6″
	g	½″φ	16	3′-9″		g	½″φ	16	3′-9″		g	½″φ	16	3′-9″		g	½″φ	16	3′-9″
	h	½″φ	4	4′-0″		h	½″φ	8	4′-0″		h	½″φ	12	4′-0″		h	½″φ	16	4′-0″
	m	½″φ	29	39′-0″		m	½″φ	29	42′-6″		m	½″φ	29	45′-6″		m	½″φ	29	48′-6″
	n	¼″□	48	12′-5″		n	¼″□	48	13′-5″		n	¼″□	48	14′-5″		n	¼″□	48	15′-5″

a. Bent transverse bars—slab. *b*. Straight transverse bars—slab. *c*. Straight longitudinal bars in parapet and curb. *d*. Top longitudinal bar in parapet. *e*. Vertical bar in parapet and curb. *f*. Vertical bar in parapet and curb. *g*. Vertical bar in parapet and curb. *h*. Vertical bar in parapet and curb. *m*. Longitudinal bar in slab-top and bottom. *n*. Longitudinal bar under I-beams.

FIG. 76*A*.—(*Continued.*)

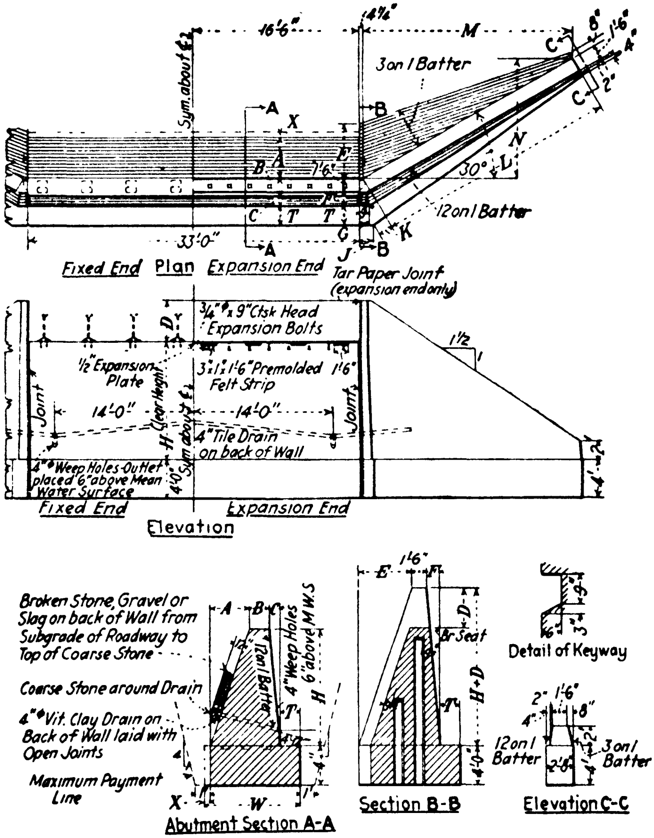
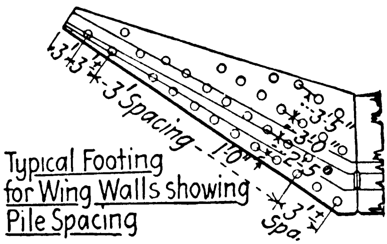


FIG. 76B.—Standard abutments steel I beam bridges. N. Y. State 1926. (Dimesions and quantities pages 286–289.)

NOTE.—Design for ordinary foundation soils. Reduce base widths if gravel, hardpan or rock is encountered.



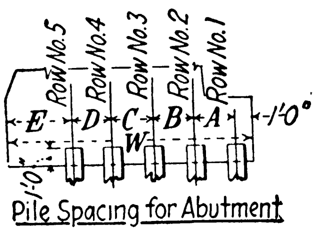


FIG. 76B.—(*Continued.*)

Clear span, feet	Clear height, H	Abutments							
		A	B	C	Footing			Toe pressure, tons sq. ft.	Cu. yds. concrete 1–2½–5
					W	T	X		
25 to 30	8′	3′– 0″	1′–6″	0′– 8″	6′– 8″	1′– 6″	.....	2.2	65.2
	10′	3′–10″	1′–6″	0′–10″	7′– 9″	1′– 7″	.....	2.3	84.9
	12′	4′– 8″	1′–6″	1′– 0″	8′–10″	1′– 8″	.....	2.5	106.9
	14′	5′– 6″	1′–6″	1′– 2″	9′–11″	1′– 9″	.....	2.6	131.2
	16′	6′– 4″	1′–6″	1′– 4″	10′–11″	1′– 9″	.....	2.8	158.0
31 to 36	10′	3′– 5″	1′–9″	0′–10″	7′– 8″	1′– 8″	.....	2.6	84.8
	12′	4′– 3″	1′–9″	1′– 0″	8′– 9″	1′– 9″	.....	2.6	107.6
	14′	5′– 2″	1′–9″	1′– 2″	10′– 0″	1′–11″	.....	2.7	133.5
	16′	6′– 1″	1′–9″	1′– 4″	11′– 2″	2′– 0″	......	2.9	164.0
	18′	7′– 0″	1′–9″	1′– 6″	12′– 3″	2′– 0″	.....	3.0	191.9
37 to 45	12′	4′– 0″	2′–0″	1′– 0″	9′– 0″	2′– 0″	.....	2.6	110.0
	14′	4′–10″	2′–0″	1′– 2″	10′– 2″	2′– 2″	.....	2.7	135.3
	16′	5′– 9″	2′–0″	1′– 4″	11′– 5″	2′– 4″	.....	2.8	164.3
	18′	6′– 8″	2′–0″	1′– 6″	12′– 9″	2′– 7″	.....	2.9	194.2
	20′	7′– 7″	2′–0″	1′– 8″	14′– 6″	2′– 9″	0′– 6″	2.7	230.3

FIG. 76B.—(*Continued.*)

Clear span, feet	Clear height, H	Wing walls										
		D	H + D	E	F	G	J	K	L	M	N	Cu. yds. concrete two wings
25 to 30	8'	2'-9½"	10'-9½"	3'- 7¼"	0'-10¾"	1'-0"	1'- 4"	0'-11¾"	15'- 2¾"	13'-2¼"	7'-7½"	47.6
	10'	2'-9½"	12'-9½"	4'- 3½"	1'- 0¾"	1'-0½"	1'- 5"	1'- 0¾"	18'- 8¼"	16'-2¼"	9'-4¼"	67.4
	12'	2'-9½"	14'-9½"	4'-11¾"	1'- 2¾"	1'-1"	1'- 5½"	1'- 1¼"	22'- 1¾"	19'-2¼"	11'-1"	92.4
	14'	2'-9½"	16'-9½"	5'- 8¼"	1'- 4¾"	1'-1½"	1'- 6¼"	1'- 2"	25'- 7½"	22'-2¼"	12'-9¾"	121.6
	16'	2'-9½"	18'-9½"	6'- 4½"	1'- 6¾"	1'-2"	1'- 6¾"	1'- 2½"	29'- 1"	25'-2¼"	14'-6½"	155.8
31 to 36	10'	3'-0"	13'-0"	4'- 4½"	1'- 1"	1'-0½"	1'- 5"	1'- 0¾"	19'- 0¾"	16'-6"	9'-6¼"	70.0
	12'	3'-0"	15'-0"	5'- 0¾"	1'- 3"	1'-1"	1'- 5¾"	1'- 1½"	22'- 6¼"	19'-6"	11'-3"	95.4
	14'	3'-0"	17'-0"	5'- 9"	1'- 5"	1'-1½"	1'- 6¾"	1'- 2½"	25'-11¾"	22'-6"	13'-0"	125.8
	16'	3'-0"	19'-0"	6'- 5¼"	1'- 7"	1'-2"	1'- 7½"	1'- 3¼"	29'- 5¼"	25'-6"	14'-8¾"	161.2
	18'	3'-0"	21'-0"	7'- 1¾"	1'- 9"	1'-2½"	1'- 8"	1'- 3¾"	32'-11"	28'-6"	16'-5½"	202.4
37 to 45	12'	3'-0"	15'-0"	5'- 0¾"	1'- 3"	1'-1"	1'- 6½"	1'- 2¼"	22'- 6¼"	10'-6"	11'-3"	96.4
	14'	3'-0"	17'-0"	5'- 9"	1'- 5"	1'-1½"	1'- 7¼"	1'- 3"	25'-11¾"	22'-6"	13'-0"	126.8
	16'	3'-0"	19'-0"	6'- 5¼"	1'- 7"	1'-2"	1'- 8¼"	1'- 4"	29'- 5¼"	25'-6"	14'-8¾"	163.0
	18'	3'-0"	21'-0"	7'- 1¾"	1'- 9"	1'-2½"	1'- 9½"	1'- 5¼"	32'-11"	28'-6"	16'-5½"	205.6
	20'	3'-0"	23'-0"	7'-10"	1'-11"	1'-3"	1'-10½"	1'- 6¼"	36'- 4½"	31'-6"	18'-2¼"	254.8

FIG. 76B.—(*Continued.*)

Span of bridge, feet	Clear height, H	Width of base, W	A	B	C	D	E	Longitudinal spacing Row 1	Row 2	Row 3	Row 4	Row 5
25 to 30	8′	6′– 8″	4′–0″	.....	.....	.....	1′– 8″	2′–6″	2′–6″			
	10′	7′– 9″	4′–0″	.....	.....	.....	2′– 9″	3′–0″	3′–0″			
	12′	8′–10″	4′–6″	.....	.....	.....	3′– 4″	2′–6″	2′–6″			
	14′	9′–11″	2′–6″	3′–6″	.....	.....	2′–11″	2′–6″	2′–6″	2′–6″		
	16′	10′–11″	2′–6″	5′–0″	.....	.....	2′– 5″	2′–6″	2′–6″	2′–6″		
31 to 36	10′	7′– 8″	4′–0″	.....	.....	.....	2′– 8″	2′–6″	2′–6″			
	12′	8′– 9″	2′–6″	2′–6″	.....	.....	2′– 9″	3′–0″	3′–0″	3′–0″		
	14′	10′– 0″	2′–6″	3′–6″	.....	.....	3′– 0″	2′–6″	2′–6″	2′–6″		
	16′	11′– 2″	2′–6″	5′–0″	.....	.....	2′– 8″	2′–6″	2′–6″	2′–6″		
	18′	12′– 3″	2′–6″	2′–6″	3′–6″	.....	2′– 9″	2′–6″	2′–6″	2′–6″	2′–6″	
37 to 45	12′	9′– 0″	2′–6″	2′–6″	.....	.....	3′–0″	3′–0″	3′–0″	3′–0″		
	14′	10′– 2″	2′–6″	3′–6″	.....	.....	3′– 2″	2′–6″	2′–6″	2′–6″		
	16′	11′– 5″	2′–6″	5′–0″	.....	.....	2′–11″	2′–6″	2′–6″	2′–6″		
	18′	12′– 0″	2′–6″	3′–0″	4′–0″	.....	2′–3″	2′–6″	2′–6″	2′–6″	2′–6″	
	20′	14′– 6″	2′–6″	2′–6″	2′–6″	2′– 6″	3′–6″	2′–6″	2′–6″	2′–6″	2′–6″	2′–6″

FIG. 76*B*.—Table of pile spacing. (Diagram page 285).—(*Continued.*)

Notes

The abutments and wing walls shown on this drawing are typical only. The engineer will give definite elevations and dimensions for each bridge.

The depth of footings shall be determined with respect to the character of the foundation material and the possibility of undermining. All footings shall rest on a firm foundation and except where rock is encountered, shall be at a depth at least four feet below the bed of stream or surface of ground.

The wing walls shall be designed and detailed to suit existing conditions and the angle between abutment and wing wall and the relative elevations of footings shall be made to fit the ground.

In case it appears that the soil will not safely withstand the unit pressure, noted in the above table of abutment dimensions, timber piles shall be used and spaced as shown in the table. These piles are designed for a maximum load of 15 tons. The tops of piles shall be placed below low water elevation and the estimated length of piles to be used shall be shown on the plans. In case excessive erosive action of the stream is expected, piles shall be used, rip-rap placed in front of the abutment or other approved means taken to prevent erosion.

Concrete in abutments and wing walls shall be 1-2½-5 mix., Item 21.

Keyways between abutments and wing walls and at all horizontal joints shall comprise about 30% of the area of surface.

All exposed edges of concrete shall be chamfered one inch.

The bases of structures shown on this sheet shall be considered as approximate only and may be ordered in writing by the Engineer to be at any elevation and of any dimensions necessary to give a proper foundation.

Payment for furnishing and placing dowels, expansion plates and bolts will be made at the respective contract prices for Metal Reinforcement and Structural Steel. (See superstructure details.)

Cost of furnishing and placing material in expansion joints at ends of bridge seat will be included in the price for Item 21.

Porous material placed on back of wall, will be paid for under Items 61, 62 and 63.

4" vitrified clay drains on back of abutments will be paid for under Item 7.

FIG. 76B.—(*Continued.*)

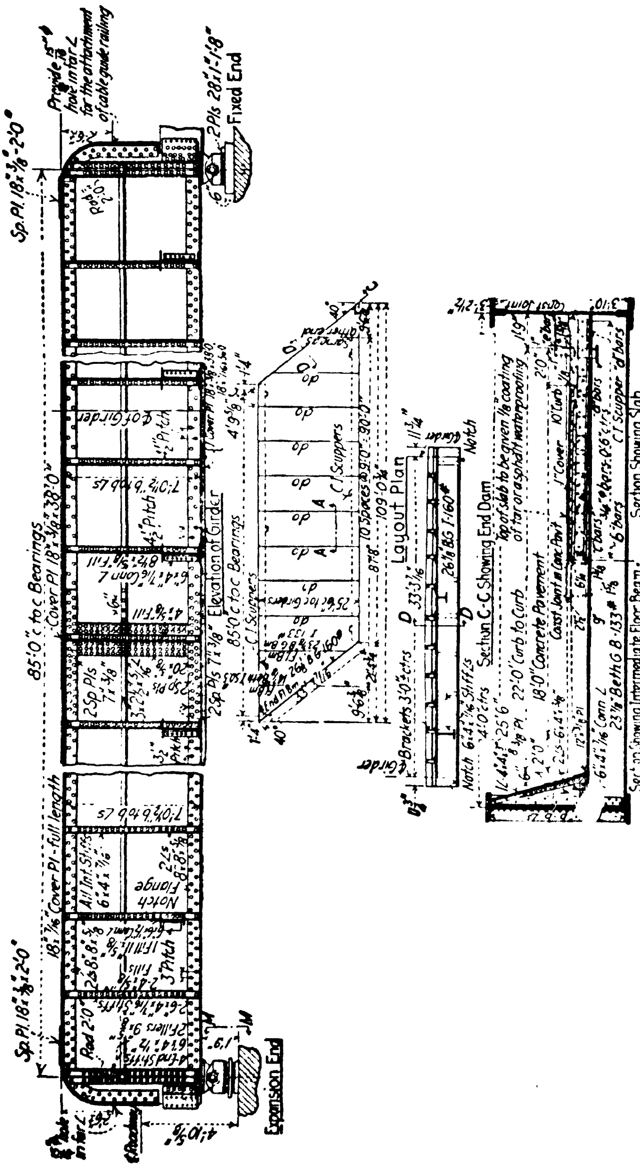


FIG. 77.—Typical plate girder. H-20 loading.

BAR LIST				
MARK	SIZE	NO	LENGTH	
a	3/4"	88	29'6"	Longitudinal Straight bars - Top and Bottom Slab
b	"	44	30'9"	3'3" — 3'11½" — 4'0" — 1'11¾" Longitud Bent bars
c	"	42	25'2"	Transverse Straight bars - Top and Bottom Slab 3"R 3"R
d	"	48	9'4"	Longitudinal Str bars under Curb-hooked both ends
e	1/2"	4	29'6"	Longitudinal bars in top of curb

NOTE: This bar list applies to 6 intermediate panels only.

FIG. 77.—(*Continued.*)

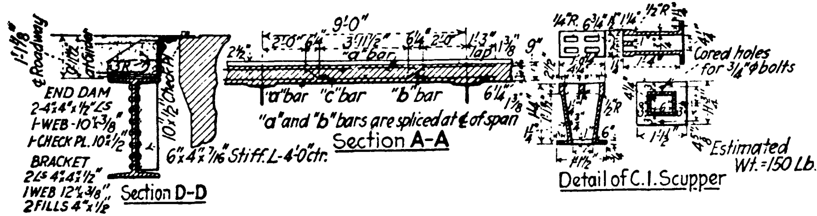


FIG. 77.—(*Continued.*)

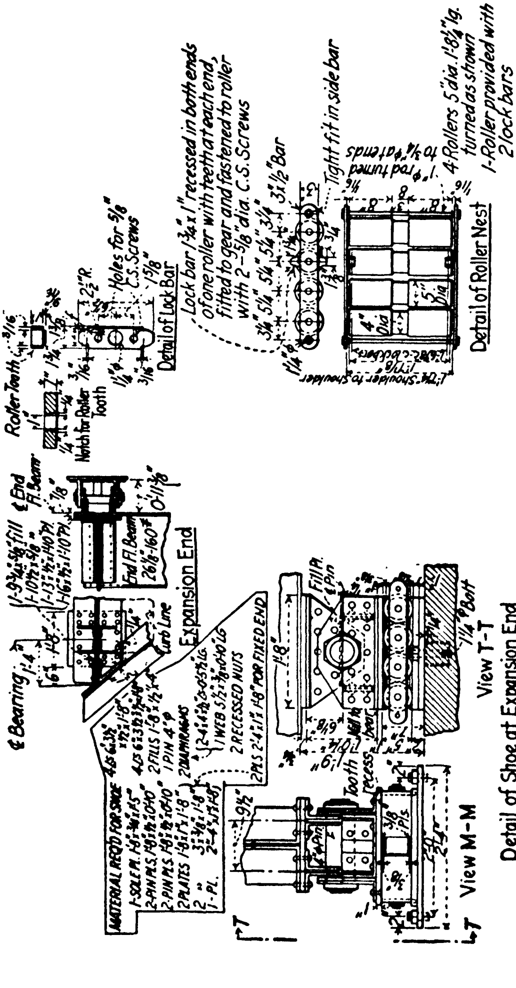


FIG. 77.—(*Continued.*)

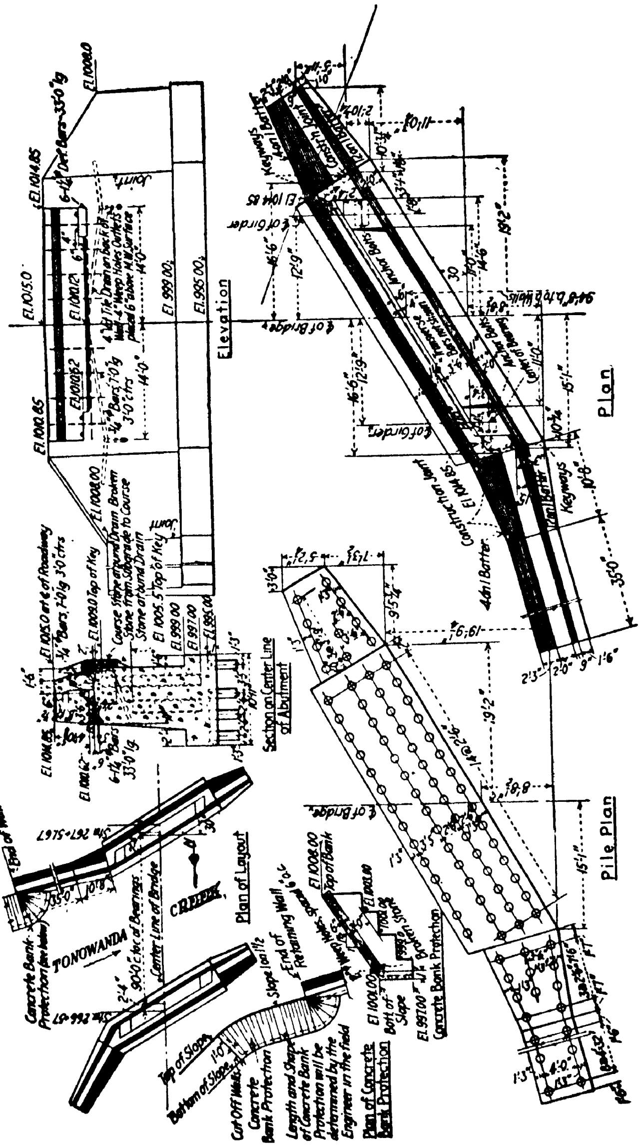
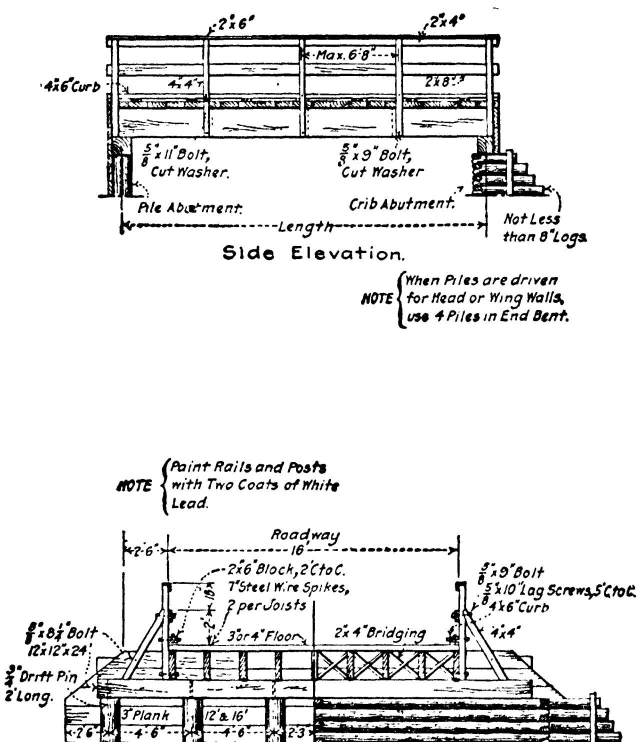


FIG. 78.—Typical plate girder abutment.



Half Section Pile Abutment. Half Section Crib Abutment.

FIG. 79.—Light wooden bridges. State of Wyoming.

BILL OF MISCELLANEOUS MATERIAL

FOR LIGHT WOODEN BRIDGES 16′ TO 25′ LONG. 16′–18′–20′ ROADWAY.
CAPACITY: 100# PER SQ. FOOT + 25% IMPACT

Name	Roadway	Maximum Spans of Joists				
		25′–0″ E. to E. 24′–0″ C. to C.	23′–0″ E. to E. 22′–0″ C. to C.	20′–0″ E. to E. 19′–0″ C. to C.	18′–0″ E. to E. 16′–9″ C. to C.	16′–0″ E. to E. 14′–6″ C. to C.
Bridging nails	Any	6#—10d	6#—10d	5#—10d	5#—10d	4#—10d
Rail nails	Any	6#—40d	6#—40d	5#—40d	5#—40d	4#—40d
Spikes for 4″ floor	16′	70#— 7″	65#— 7″	57#— 7″	51#— 7″	45#— 7″
Spikes for 4″ floor	18′	78#— 7″	72#— 7″	63#— 7″	57#— 7″	50#— 7″
Spikes for 4″ floor	20′	86#— 7″	80#— 7″	70#— 7″	62#— 7″	55#— 7″
Spikes for 3″ floor	16′	50#— 6″	45#— 6″	40#— 6″	55#— 6″	32#— 6″
Spikes for 3″ floor	18′	55#— 6″	50#— 6″	45#— 6″	40#— 6″	36#— 6″
Spikes for 3″ floor	20′	60#— 6″	58#— 6″	50#— 6″	45#— 6″	40#— 6″
Curb lag screws	Any	12—5/8″×10″	12—5/8″×10″	10—5/8″×10″	10—5/8″×10″	8—5/8″×10″
Rail bolts	Any	14—5/8″× 9″	14—5/8″× 9″	12—5/8″× 9″	12—5/8″× 9″	12—5/8″× 9″
Rail bolts	Any	10—5/8″×11″	10—5/8″×11″	8—5/8″×11″	8—5/8″×11″	8—5/8″×11″
Rail bolts	Any	4—5/8″×8¼″	4—5/8″×8¼″	4—5/8″×8¼″	4—5/8″×8¼″	4—5/8″×8¼″
Pressed washers	Any	74 for 5/8″ bolts	74 for 5/8″ bolts	64 for 5/8″ bolts	64 for 5/8″ bolts	62 for 5/8″ bolts

ALL NAILS AND SPIKES TO BE STEEL WIRE

FIG. 79.—(*Continued.*)

MATERIAL FOR PILE ABUTMENT

Name	Roadway	Quantity	Size	Length or Kind
Piling	16'–18'	6	12" butt dia.	10'–0" in ground
Cap	Any	1	12" × 12"	24'–0"
Backing	Any	6 or more	3" × 12"	12'–0"
Backing	Any	6 or more	3" × 12"	16'–0"
Drifts	Any	6	¾" φ	2'–0"
Nails for backing plk	Any	7½#	60d.	Steel wire nails

Maximum Length C. to C.	Roadway			Maximum Wheel Load Assumed on 2 Joists
	16'–0"	18'–0"	20'–0"	
24'–6"	9—4" × 18"	10—4" × 18"	11—4" × 18"	3600#
22'–0"	9—4" × 16"	10—4" × 16"	11—4" × 16"	5000#
19'–3"	9—4" × 14"	10—4" × 14"	11—4" × 14"	4350#
16'–9"	9—4" × 12"	10—4" × 12"	11—4" × 12"	3700#
19'–3"	9—3" × 16"	10—3" × 16"	11—3" × 16"	4250#
16'–9"	9—3" × 14"	10—3" × 14"	11—3" × 14"	3750#
14'–6"	9—3" × 12"	10—3" × 12"	11—3" × 12"	3000#
UNIT LOAD: 100# per sq. foot + 25% impact				Impact 25% allowed for

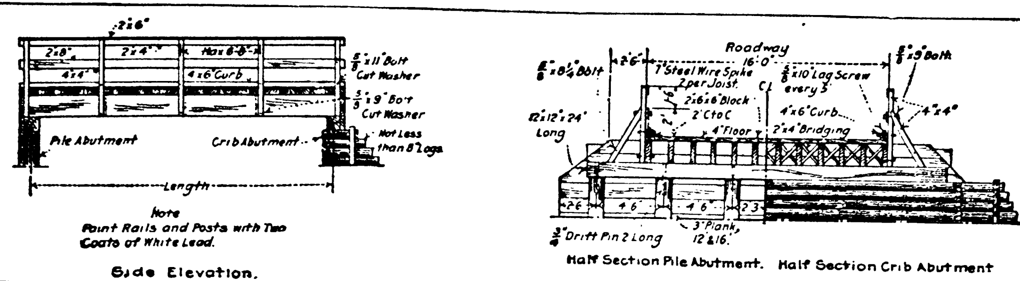
3" × 12" Floor (assumed worn to 2") Good for 1600# Conc. Load
4" × 12" Floor (assumed worn to 3") Good for 3600# Conc. Load

FIG. 79.—(*Continued.*)

BILL OF LUMBER
FOR LIGHT WOODEN BRIDGES 16′ TO 25′ LONG. 16′–18′–20′ ROADWAY.
CAPACITY: 100# PER SQ. FOOT + 25% IMPACT

Name	Maximum Spans for Joists Shown				
	25′–0″ E. to E. 24′–0″ C. to C.	23′–0″ E. to E. C. to C.	20′–0″ E. to E. 19′–0″ C. to C.	18′–0″ E. to E. 16′–9″ C. to C.	16′–0″ E. to E. 14′–6″ C. to C.
			16′–0″ Roadway		
Joists	9—4″×18″×25′	9—4″×16″×23′	9—3″×16″×20′ or 9—4″×14″×20′	9—3″×14″×18′ or 9—4″×12″×18′	9—3″×12″×16′
Floor	27—3″×12″×16′ or 27—4″×12″×16′	25—3″×12″×16′ or 25—4″×12″×16′	22—3″×12″×16′ or 22—4″×12″×16′	20—3″×12″×16′ or 20—4″×12″×16′	18—3″×12″×16′ or 18—4″×12″×16′
Bridging	3—2″×4″×16′	3—2″×4″×16′	2—2″×4″×16′	2—2″×4″×16′	2—2″×4″×16′
			18′–0″ Roadway		
Joists	10—4″×18″×25′	10—4″×16″×23′	10—3″×16″×20′ or 10—4″×14″×20′	10—3″×14″×18′ or 10—4″×12″×18′	10—3″×12″×16′
Floor	27—3″×12″×18′ or 27—4″×12″×18′	25—3″×12″×18′ or 25—4″×12″×18′	22—3″×12″×18′ or 22—4″×12″×18′	20—3″×12″×18′ or 20—4″×12″×18′	18—3″×12″×16′ or 18—4″×12″×16′
Bridging	4—2″×4″×14′	4—2″×4″×14′	3—2″×4″×12′	3—2″×4″×12′	3—2″×4″×12′
			20′–0″ Roadway		
Joists	11—4″×18″×25′	11—4″×16″×23′	11—3″×16″×20′ or 11—4″×14″×20′	11—3″×14″×18′ or 11—4″×12″×18′	11—3″×12″×16′
Floor	27—3″×12″×20′ or 27—4″×12″×20′	25—3″×12″×20′ or 25—4″×12″×20′	22—3″×12″×20′ or 22—4″×12″×20′	20—3″×12″×20′ or 20—4″×12″×20′	18—3″×12″×20′ or 18—4″×12″×20′
Bridging	4—2″×4″×16′	4—2″×4″×16′	3—2″×4″×14′	3—2″×4″×14′	3—2″×4″×14′
			Any Roadway		
Curb	3—4″×6″×16′	3—4″×6″×16′	3—4″×6″×14′	2—4″×6″×18′	2—4″×6″×16′
Rail S.4S.	5—4″×4″×16′ 4—2″×4″×14′ 4—2″×6″×14′ 4—2″×8″×14′	5—4″×4″×16′ 4—2″×4″×12′ 4—2″×6″×12′ 4—2″×8″×12′	4—4″×4″×16′ 2—2″×4″×20′ 2—2″×6″×20′ 2—2″×8″×20′	4—4″×4″×18′ 2—2″×4″×18′ 2—2″×6″×18′ 2—2″×8″×18′	4—4″×4″×16′ 2—2″×4″×16′ 2—2″×6″×16′ 2—2″×8″×16′
Blocks	27—2″×6″×6″	25—2″×6″×6″	22—2″×6″×6″	20—2″×6″×6″	18—2″×6″×6″

FIG. 79.—(*Continued.*)



Bill of Miscellaneous Material

For Standard Wooden Bridges 8' to 20' Long. 16'-18'-20' Roadway.
Capacity: 20-ton Traction Eng. (⅔ on Rear Axle) + 25% Impact

Name	Roadway	Maximum Spans of Joists: 20'-0" E. to E. 19'-0" C. to C.	16'-0" E. to E. 15'-0" C. to C.	12'-0" E. to E. 11'-0" C. to C.	10'-0" E. to E. 9'-0" C. to C.	8'-0" E. to E. 7'-0" C. to C.
Bridging nails	Any	6#—10d	6#—10d	4#—10d	4#—10d	
Rail nails	Any	5#—40d	4#—40d	3#—40d	3#—40d	2#—40d
Floor spikes	16	80#— 7"	64#— 7"	48#— 7"	40#— 7"	32#— 7"
Floor spikes	18	80#— 7"	64#— 7"	48#— 7"	40#— 7"	32#— 7"
Floor spikes	20	88#— 7"	70#— 7"	53#— 7"	44#— 7"	35#— 7"
Curb lag screws	Any	10—⅝"×10"	8—⅝"×10"	6—⅝"×10"	6—⅝"×10"	6—⅝"×10"
Rail bolts	Any	12—⅝"× 9"	12—⅝"× 9"	10—⅝"× 9"	10—⅝"× 9"	8—⅝"× 9"
Rail bolts	Any	8—⅝"×11"	8—⅝"×11"	6—⅝"×11"	6—⅝"×11"	4—⅝"×11"
Rail bolts	Any	4—⅝"×8¼"	4—⅝"×8¼"	4—⅝"×8¼"	4—⅝"×8¼"	4—⅝"×8¼"
Pressed washers	Any	64 for ⅝" bolts	62 for ⅝" bolts	50 for ⅝" bolts	50 for ⅝" bolts	42 for ⅝" bolts

All Nails and Spikes to be Steel Wire

Fig. 80.

Material for One Pile Abutment

Name	Roadway	Quantity	Size	Length or Kind
Piling	16′–18′	6	12″ butt dia.	10′–0″ in ground
Cap	Any	1	12″ × 12″	24′–0″
Backing	Any	6 or more	3″ × 12″	12′–0″
Backing	Any	6 or more	3″ × 12″	16′–0″
Drifts	Any	6	¾″ φ	2′–0″
Nails for backing plk.	Any	7½#	60d	Steel wire nails

Maximum Length C. to C.	Roadway		
	16′–0″	18′–0″	20′–0″
20′–0″	12—4″ × 18″	12—4″ × 18″	14—4″ × 18″
16′–0″	12—4″ × 16″	12—4″ × 16″	14—4″ × 16″
12′–0″	12—4″ × 14″	12—4″ × 14″	14—4″ × 14″
11′–9″	12—3″ × 16″	12—3″ × 16″	14—3″ × 16″
9′–0″	12—3″ × 14″	12—3″ × 14″	14—3″ × 14″
8′–9″	12—4″ × 12″	12—4″ × 12″	14—4″ × 12″
6′–6″	12—3″ × 12″	12—3″ × 12″	14—3″ × 12″

Fig. 80.—(*Continued.*)

Bill of Lumber
For Standard Wooden Bridges 8' to 20' Long. 16'–18'–20' Roadway.
Capacity: 20-ton Traction Eng. (3/5 on Rear Axle) + 25% Impact

Name	Maximum Spans for Joists Shown				
	20'–0" E. to E. 19'–0" C. to C.	16'–0" E. to E. 15'–0" C. to C.	12'–0" E. to E. 11'–0" C. to C.	10'–0" E. to E. 9'–0" C. to C.	8'–0" E. to E. 7'–0" C. to C.
16'–0" Roadway					
Joists	12—4"×18"×20'	12—4"×16"×16'	12—3"×16"×12' or 12—4"×14"×12'	12—3"×14"×10' or 12—4"×12"×10'	12—3"×12"×8'
Floor	22—4"×12"×16'	18—4"×12"×16'	14—4"×12"×16'	11—4"×12"×16'	9—4"×12"×16'
Bridging	9—2"×4"×16'	9—2"×4"×16'	5—2"×4"×16'	5—2"×4"×16'	
18'–0" Roadway					
Joists	12—4"×18"×20'	12—4"×16"×16'	12—3"×16"×12' or 12—4"×14"×12'	12—3"×14"×10' or 12—4"×12"×10'	12—3"×12"×8'
Floor	22—4"×12"×18'	18—4"×12"×18'	14—4"×12"×18'	11—4"×12"×18'	9—4"×12"×18'
Bridging	10—2"×4"×16'	10—2"×4"×16'	6—2"×4"×16'	6—2"×4"×16'	
20'–0" Roadway					
Joists	14—4"×18"×20'	14—4"×16"×16'	14—3"×16"×12' or 14—4"×14"×12'	14—3"×14"×10' or 14—4"×12"×10'	14—3"×12"×8'
Floor	22—4"×12"×20'	18—4"×12"×20'	14—4"×12"×20'	11—4"×12"×20'	9—4"×12"×20'
Bridging	12—2"×4"×16'	12—2"×4"×16'	7—2"×4"×16'	7—2"×4"×16'	
Any Roadway					
Curb	3—4"×6"×14'	2—4"×6"×16'	2—4"×6"×12'	2—4"×6"×10'	2—4"×6"×8'
Rail S. 4S.	4—4"×4"×16' 2—2"×4"×20' 2—2"×6"×20' 2—2"×8"×20'	4—4"×4"×16' 2—2"×4"×16' 2—2"×6"×16' 2—2"×8"×16'	3—4"×4"×16' 2—2"×4"×12' 2—2"×6"×12' 2—2"×8"×12'	3—4"×4"×16' 2—2"×4"×10' 2—2"×6"×10' 2—2"×8"×10'	2—4"×4"×16' 2—2"×4"×8' 2—2"×6"×8' 2—2"×8"×8'
Blocks	22—2"×6"×6"	18—2"×6"×6"	14—2"×6"×6"	12—2"×6"×6"	10—2"×6"×6"

Fig. 80.—(*Continued.*)

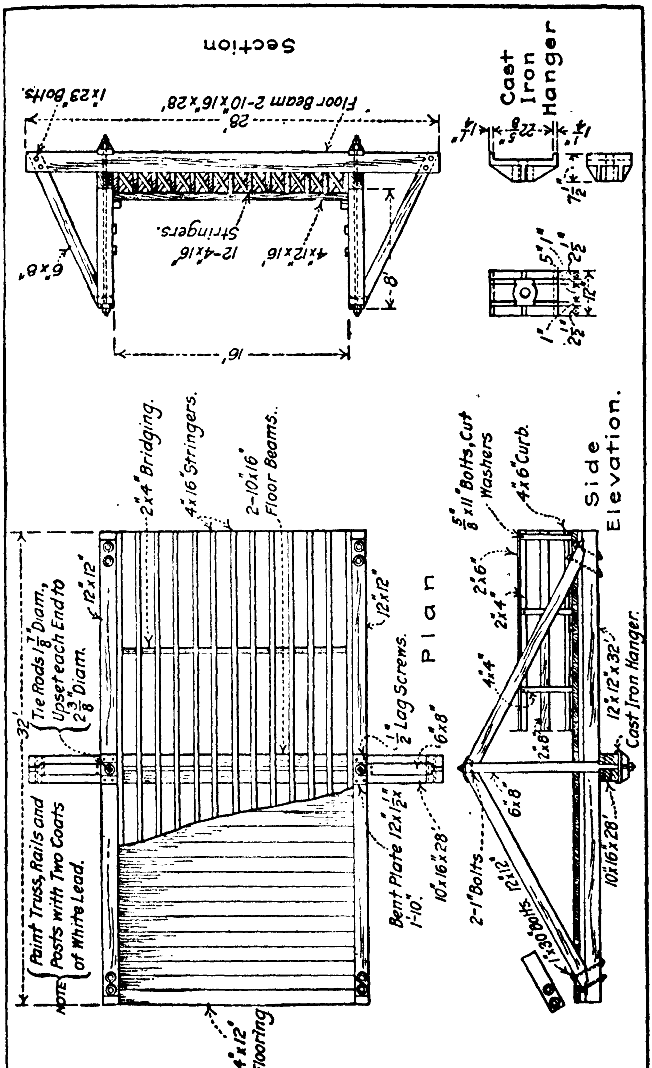


FIG. 81.—30′ span king post timber bridge. State of Wyoming.

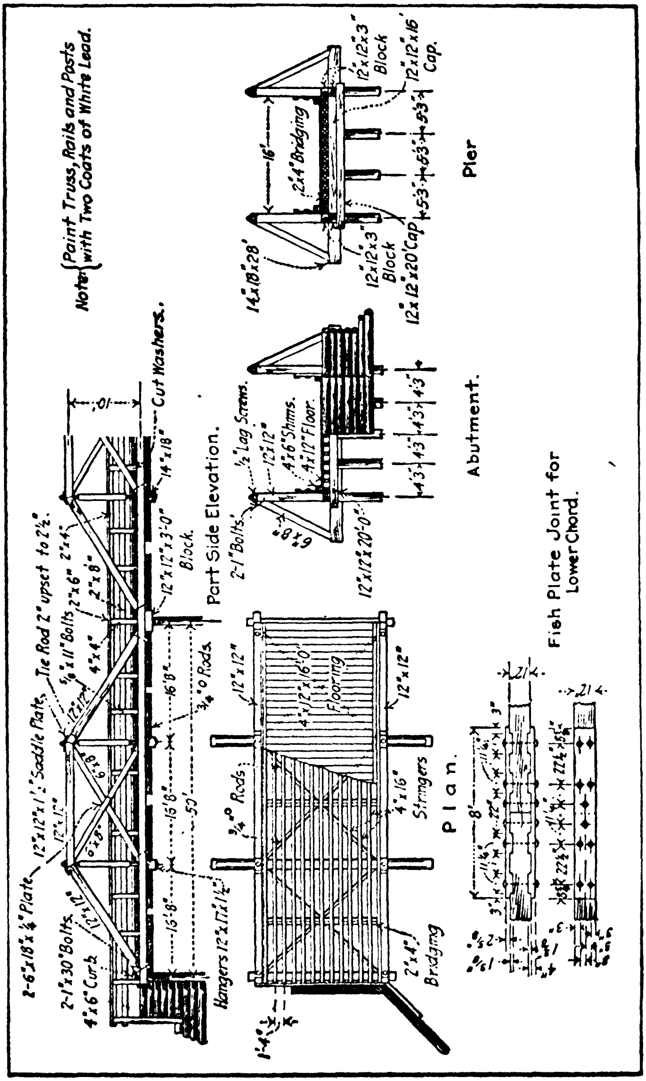


FIG. 82.—Typical 50' span pony truss timber bridge. State of Wyoming.

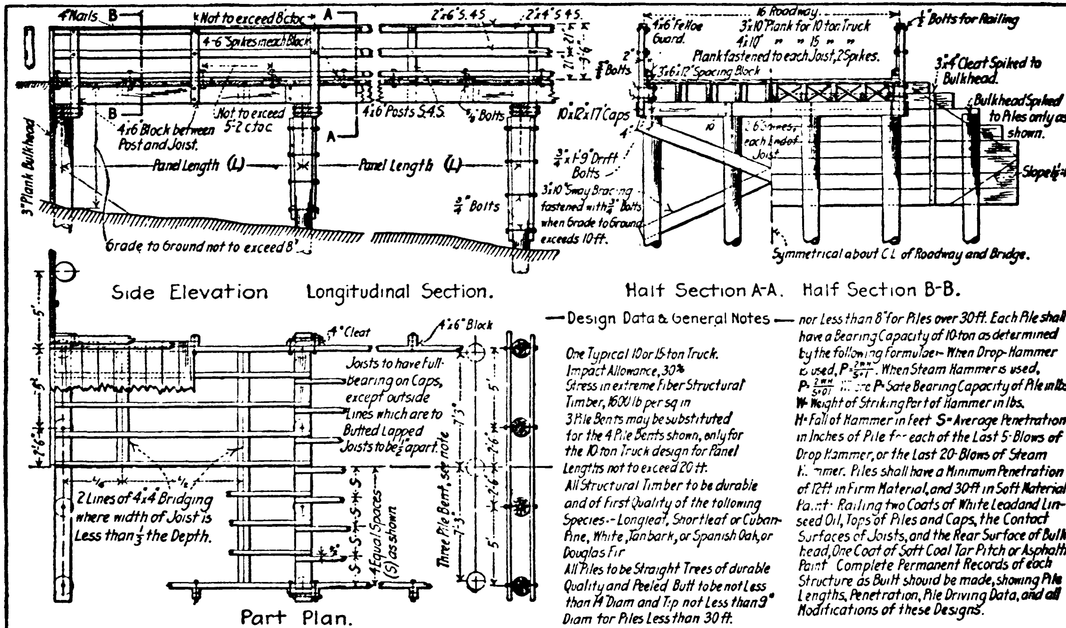


FIG. 83.—Typical pile trestle. U. S. office of public roads.

Panel Length (L)	Intermediate Panel				
	Size of Joists	Joists	Floor Railing Details	Total Lumber	Bolts, Washers, Spikes, Nails
Feet	Inches	Ft. B. M.	Ft. B. M.	Ft. B. M.	Pounds
10	6×12	590	800	1390	80
	4×14	460	840	1300	
11	6×12	650	870	1520	90
	4×14	500	920	1420	
12	6×12	700	940	1640	90
	4×16	620	990	1610	
	8×12	1010	1020	2030	
13	6×14	880	1020	1900	90
	4×16	670	1070	1740	
	8×12	1080	1090	2170	
14	6×14	950	1090	2040	90
	4×16	720	1140	1860	
15	8×12	1150	1170	2320	100
	6×14	1010	1170	2180	
16	10×12	1530	1240	2770	
	6×14	1070	1240	2310	100
	6×16	1230	1240	2470	
17	10×12	1620	1340	2960	
	8×14	1510	1340	2850	120
	6×16	1300	1340	2640	
18	10×12	1710	1410	3120	
	8×14	1600	1410	3010	130
	6×16	1370	1410	2780	
19	10×12	1800	1490	3290	
	8×14	1680	1490	3170	130
	6×16	1440	1490	2930	
20	8×14	1760	1560	3320	130
	8×16	2020	1560	3580	
21	10×14	2310	1640	3950	140
	8×16	2110	1640	3750	
22	10×14	2410	1710	4120	150
	8×16	2210	1710	3920	

FIG. 83.—(*Continued.*)

Panel Length (L)	Size of Joists	Intermediate Panel			
		Joists	Floor Railing Details	Total Lumber	Bolts, Washers, Spikes, Nails
Feet	Inches	Ft. B. M.	Ft. B. M.	Ft. B. M.	Pounds
10	4×12	400	640	1040	70
	3×14	350	680	1030	
11	4×12	430	700	1130	80
	3×14	380	740	1120	
12	6×12	700	760	1460	80
	3×14	410	800	1210	
13	6×12	760	810	1570	80
	4×14	590	860	1450	
14	6×12	810	870	1680	
	4×14	630	910	1540	90
	4×16	720	910	1630	
15	6×12	860	930	1790	
	4×14	670	970	1640	90
	4×16	770	970	1740	
16	6×12	920	990	1910	
	6×14	1070	990	2060	100
	4×16	820	1030	1850	
17	6×12	970	1070	2040	
	6×14	1130	1070	2200	110
	4×16	870	1110	1980	
18	8×12	1370	1130	2500	
	6×14	1200	1130	2330	120
	4×16	910	1170	2080	
19	8×12	1440	1180	2620	
	6×14	1260	1180	2440	120
	4×16	960	1220	2180	
20	8×12	1510	1240	2750	
	6×14	1320	1240	2560	120
	6×16	1510	1240	2750	
21	10×12	1980	1300	3280	
	6×14	1390	1300	2690	130
	6×16	1580	1300	2880	
22	10×12	2070	1360	3430	
	8×14	1930	1360	3290	130
	6×16	1660	1360	3020	
23	10×12	2160	1420	3580	
	8×14	2020	1420	3440	130
	6×16	1730	1420	3150	
24	10×12	2250	1470	3720	
	8×14	2100	1470	3570	130
	6×16	1800	1470	3270	
25	10×14	2730	1560	4290	
	8×14	2180	1560	3740	150
	6×16	1870	1560	3430	
26	10×14	2840	1610	4450	160
	8×16	2600	1610	4210	
27	10×14	2940	1670	4610	160
	8×16	2690	1670	4360	
28	10×14	3050	1730	4780	160
	8×16	2780	1730	4510	
29	10×14	3150	1790	4940	160
	8×16	2880	1790	4670	

Washers to be ogee type cast iron ⅝″ and ⅝″ bolts, and cut wrought iron or steel plate washers for ½″ bolts.

FIG. 83.—(*Continued.*)

Grade to Ground	Sway Bracing—Intermediate Bent			
	Sets	Length	Lumber	Bolts
Feet	No. Reqd.	Feet	Ft. B. M.	Pounds
10–12	1	18	90	35
12–15	1	20	100	35
15–18	1	22	110	35
18–23	2	18 & 20	190	60
23–26	2	20	200	60
One cap 10″×12″×17′-0″			170	10

Grade to Ground	Bulkhead—End Bent	
	Lumber	Spikes
Feet	Ft. B. M.	Pounds
4	270	5
5	360	5
6	460	10
7	550	10
8	640	10

FIG. 83.—(*Continued.*)

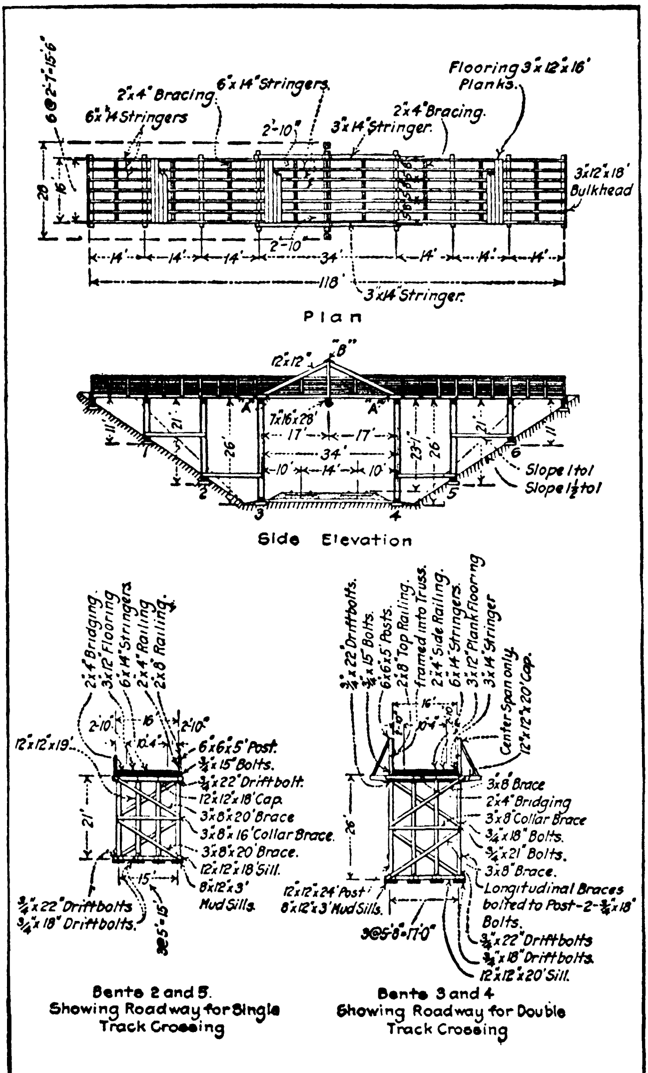


FIG. 84.—Typical framed trestle. Illinois Central Railroad.

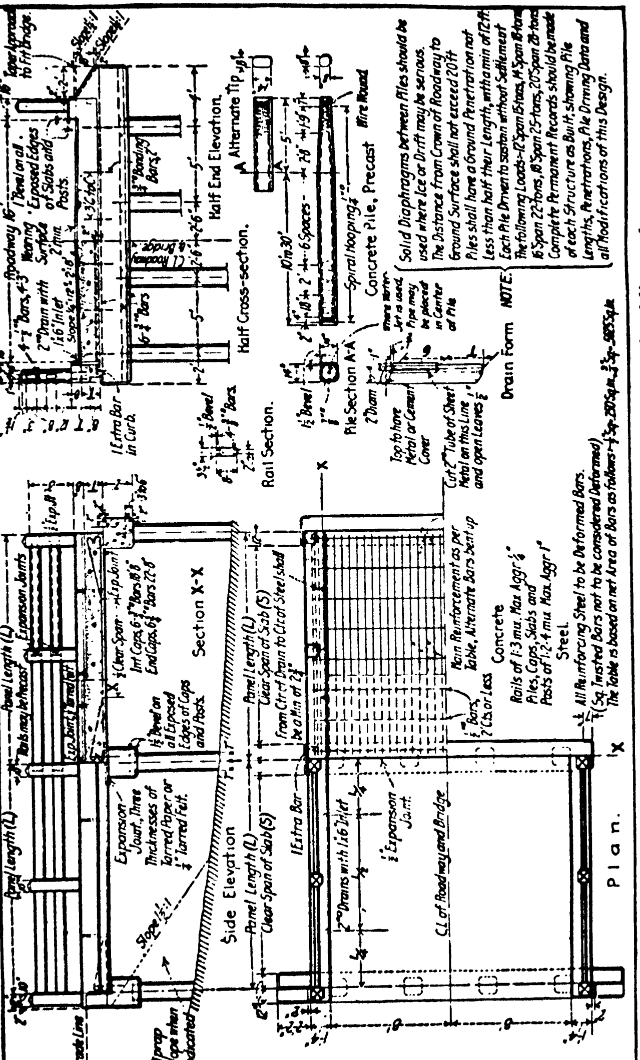


FIG. 85.—Typical concrete pile trestle. U. S. office of public roads.

Item	Panel Length	Clear Span	Min. Slab Thickness	Reinforcing Steel	Intermediate Panel		End Panel	
					Concrete	Steel	Concrete	Steel
	L	S	T	Main Reinforcement	Cubic Yards	Pounds	Cubic Yards	Pounds
Slabs and curbs...	14 Ft.	12 Ft.	12½ In.	¾" sq. 7" ctrs.	11.46	1070	12.29	1070
	16 Ft.	14 Ft.	14 In.	¾" sq. 6" ctrs.	14.48	1370	15.40	1370
	18 Ft.	16 Ft.	16 In.	⅞" sq. 7½" ctrs.	18.38	1680	19.41	1680
	20 Ft.	18 Ft.	17½ In.	⅞" sq. 6½" ctrs.	22.14	2130	23.26	2130
	22 Ft.	20 Ft.	19½ In.	⅞" sq. 6" ctrs.	26 89	2460	28.13	2460
Railing.........	One rail post				0.0762 Cu. Yd. Concr.		14.45 Lb. Steel	
	One rail (section 3½" × 8") per lin. ft.				0.0072 Cu. Yd. Concr.		1.913 Lb. Steel	
Bents............	One pile 15 feet long				0.67 Cu. Yd. Concr.		174.0 Lb. Steel	
	One pile (section 1'-2" × 1'-2") per lin. ft.				0.0504 Cu. Yd. Concr.		11.6 Lb. Steel	
	One cap for intermediate bent				2.82 Cu. Yd. Concr.		238.0 Lb. Steel	
	One cap for end bent with walls				3.55 Cu. Yd. Concr.		283.0 Lb. Steel	

DESIGN DATA:
Steel in tension, 16,000 lb. per sq. in.
Concrete in compression 600 lb. per sq. in.
Concentrated load, one 15-ton typical truck.
Impact allowance, 30%.
Paving not to exceed 120 lb. per sq. ft.

FIG. 85.—(*Continued.*)

Quick-estimating Diagrams.—The following quick-estimating diagrams are convenient in determining economic type and rough appropriation estimates. The unit prices used are based on data in Chap. IX (p. 661). Weights of steel, if desired, can be obtained from Chap. IX (pp. 649 to 654).

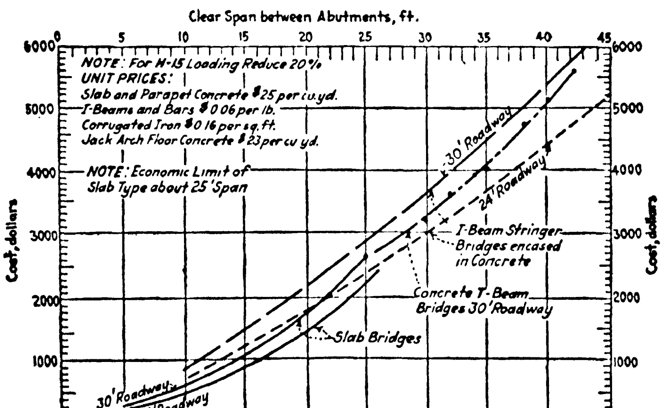


FIG. 86.—Typical cost curves. Small span bridge superstructures. H-20 loading. Based on standard bridges in the following illustrations. Slabs, Fig. 69, page 237. Steel I beam stringers, Fig. 76, page 275. Concrete T beam stringers, Fig. 73, page 252.

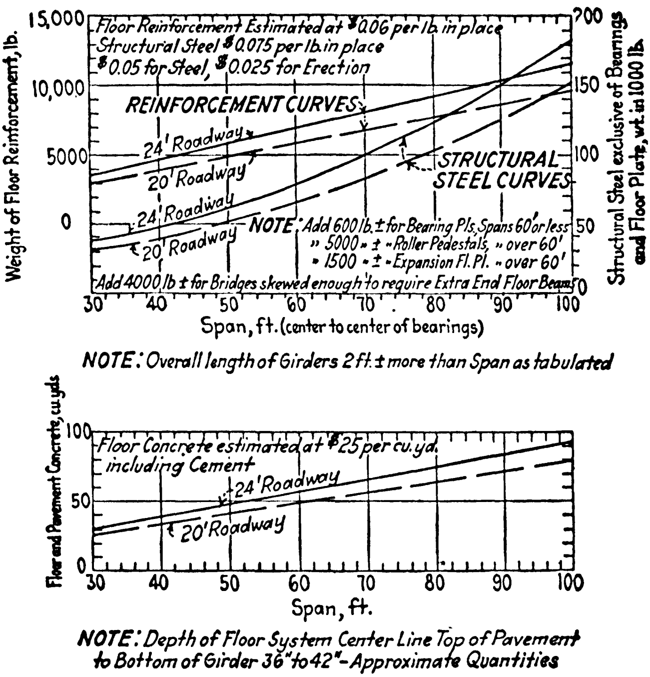


Fig. 87.—Approximate quantities typical plate girders. H-20 loading. Structural steel includes girders, floor beams and stringers.

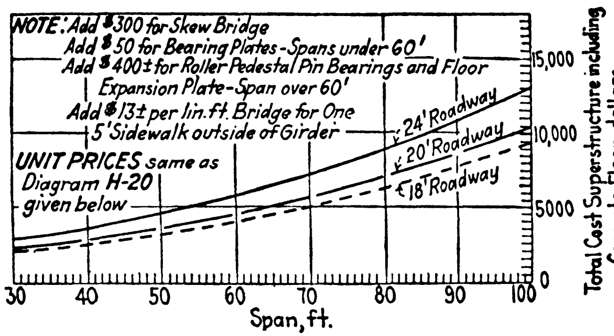


FIG. 88A.—Typical cost curves. Plate girder superstructures. H-15 loading.

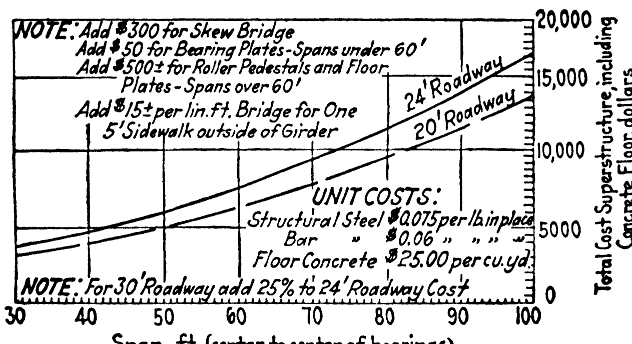


FIG. 88B.—Typical cost curves. Plate girder superstructures H-20 loading.

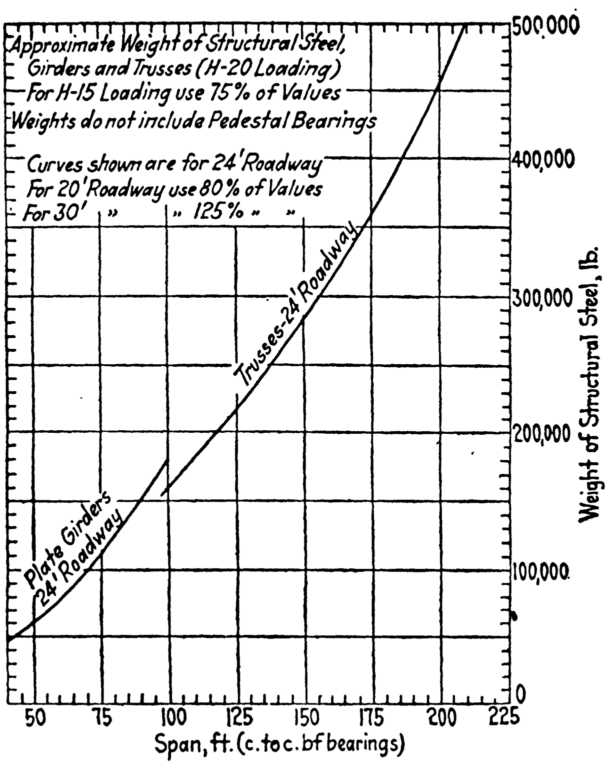


FIG. 89.—Approximate quantities structural steel plate girders and trusses. H-20 loading. These weights include girder or trusses, floor beams and stringers.

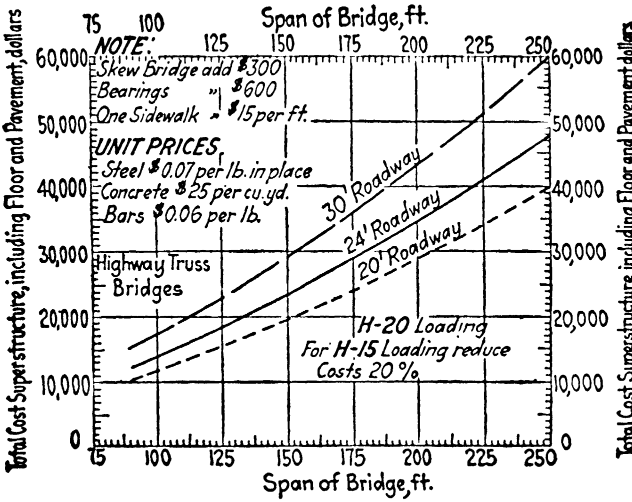


FIG. 90.—Typical cost curves. Truss bridge superstructures complete. H-20 loading.

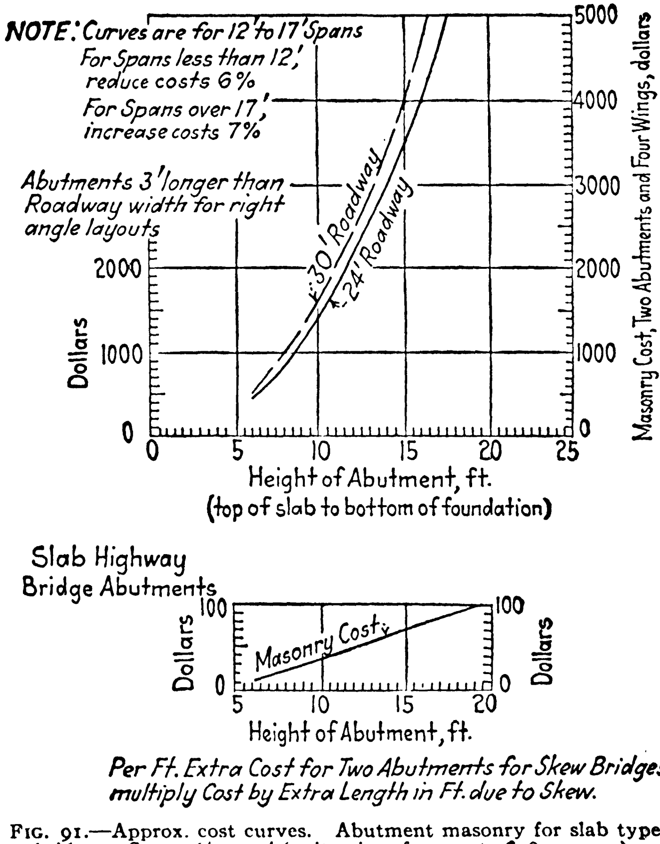


FIG. 91.—Approx. cost curves. Abutment masonry for slab type bridges. Spans 6' to 25' (unit price of concrete $18 per c.y.).

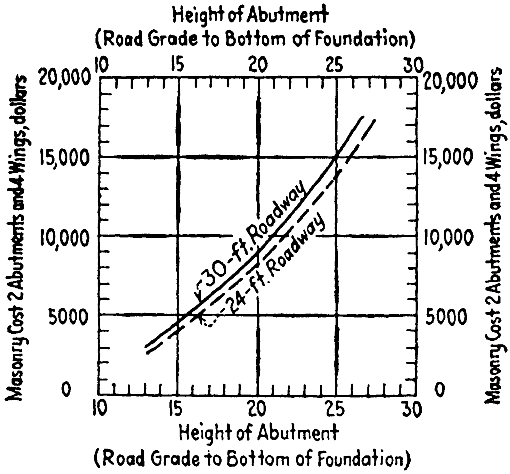


FIG. 91*A*.—Approximate cost abutment masonry. 2 abutments and 4 wings (rt. angle spans). I beam stringer bridges 25 to 45′ span (concrete $18 per c.y.).

APPROXIMATE COST OF MASONRY IN BRIDGE PIERS PER FT. LENGTH OF PIER SHORT SPAN BRIDGES (Concrete at $18 per c. y.)

Height of pier road grade to bottom of foundations in feet	Length of spans between piers			
	10′	20′	30′	40′
10	$ 8.50	$14.00	$18.00	$20.00
11	9.30	15.50	20.00	22.70
12	10.00	17.00	22.00	25.50
13	11.00	18.50	24.00	28.20
14		20.00	26.00	30.80
15		21.50	28.00	33.50
16			30.00	36.20
17			32.50	39.00
18			35.00	42.00
19				45.00
20				48.00

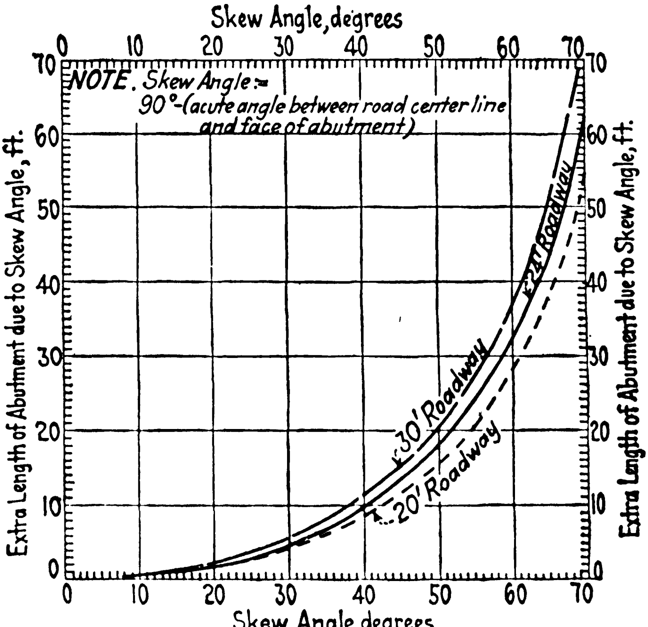


FIG. 92.—Extra length of abutment due to skew angle of bridge

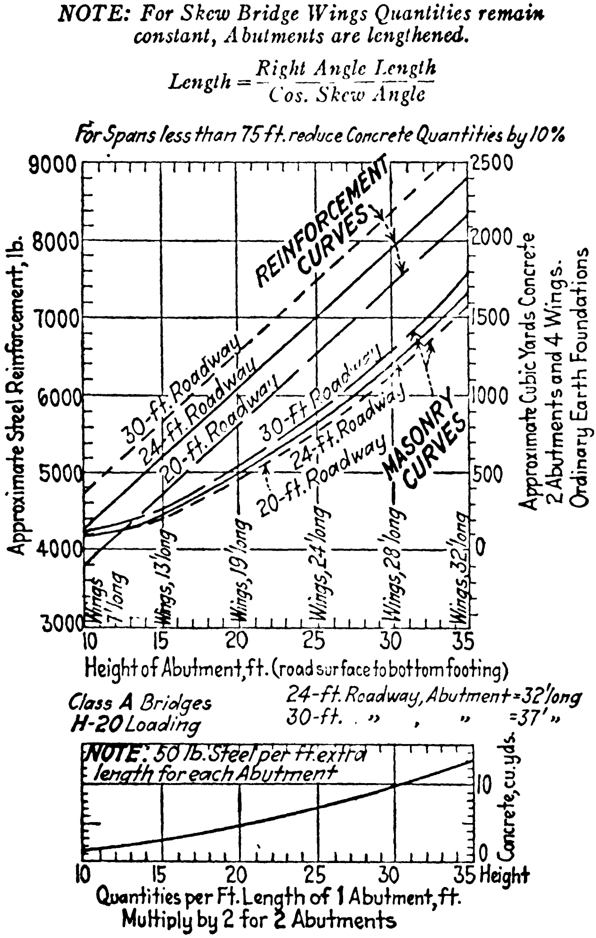


FIG. 93.—Approximate quantities of abutment masonry plate girder. Type of bridge. 40′ to 100′ spans.

GENERAL SPECIFICATIONS, PLATE GIRDER BRIDGE ABUTMENTS

24′ roadway girders 27 C. to C.
32′ length of abutment for rt.< layout.
Width bridge seat.
30 to 50′ spans........................... 2.5 feet
50 to 100′ spans.......................... 3.0 feet
Back Walls. 18″ wide on top.
24″ wide at elev. bridge seat.
Spans 30 to 60 feet.
30″ wide at elev. of bridge seat.
Spans 60 to 100′ where roller bearings are used.
Top of backwall level with bottom of bridge floor at fixed end.
Top of backwall at expansion end level with top of floor and notched for floor plate and for approach pavement.
Back walls reinforced vertically and horizontally
Wings 24″ wide on top + 3″ for coping offset.
Bridge seat 18″ thick + 3″ coping offset.
Face batter 1″ to 2″ in 12″.
Footing course at least 4′ below stream bed.
Toe extension as needed. Normally 2 ft.
Width of abutment at top of footing course at least 0.4 h and preferably 0.45 h.
Width of abutment at bottom of footing course normally 0.5 h.
Rankine's earth pressure formula used in developing base pressure diagrams.

DIAGRAMS

Use equivalent live load surcharge of 1.5 feet. in computing foundation pressure.

MAXIMUM FOUNDATION PRESSURE

Pile loads	15 tons per pile
Soil loads per sq. ft.	
Silt	0.5 tons
Loam	1.0 tons
Stiff clay	3–4
Mixture clay and gravel	2
Firm gravel	4–5
Hardpan	6–7
Rock	8–20

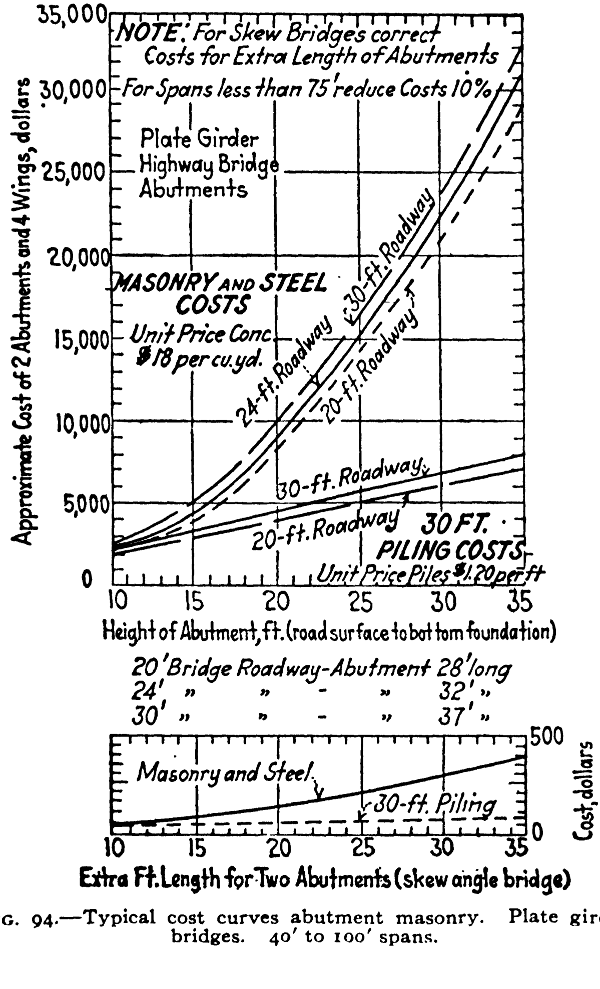


FIG. 94.—Typical cost curves abutment masonry. Plate girder bridges. 40′ to 100′ spans.

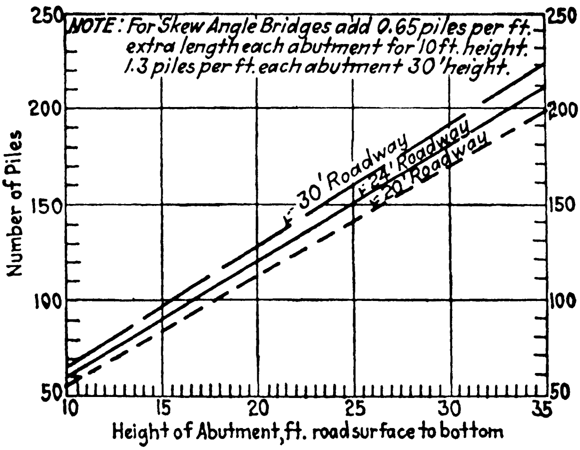


FIG. 95.—Approx. number of piles under girder abutments.

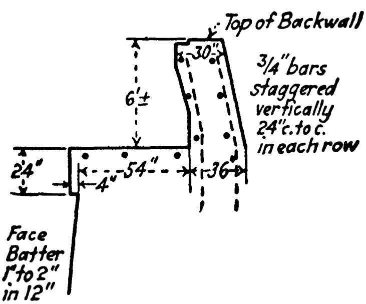


FIG. 96.—Typical seat and backwall long span truss bridge.

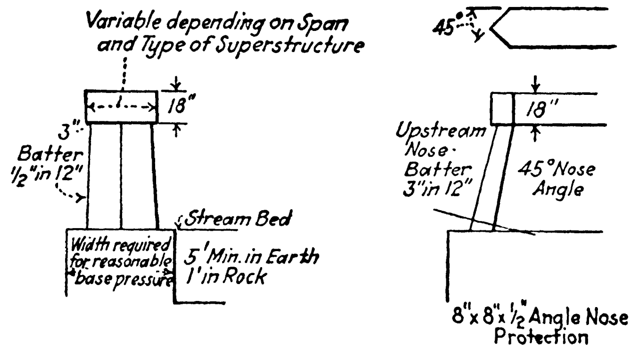


FIG. 97.—General dimensions of piers.

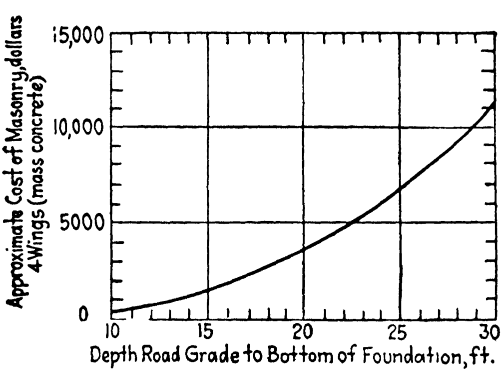


FIG. 98.—Approximate cost concrete arch bridge wings. Concrete at $20 per cubic yard.

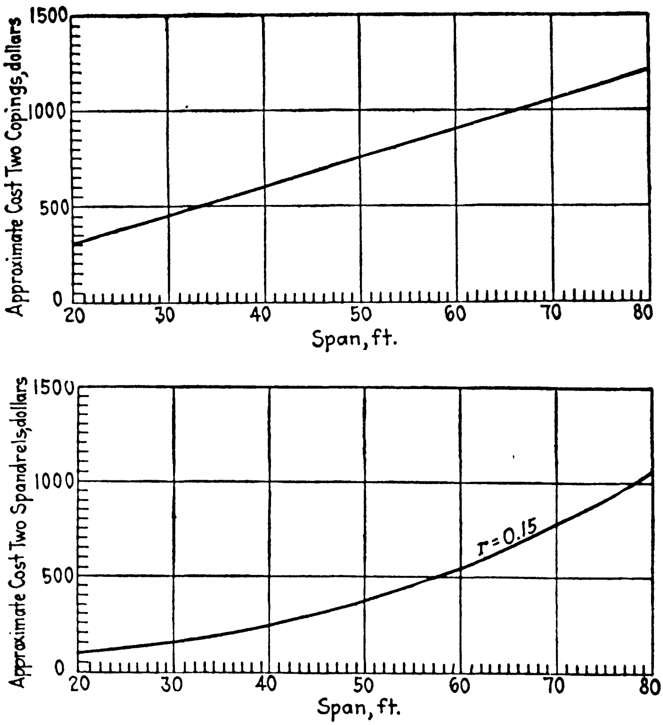


FIG. 99.—Approximate cost concrete arch bridge spandrels and copings. Rise ratio 0.15. (Concrete estimated $35 per c.y.)

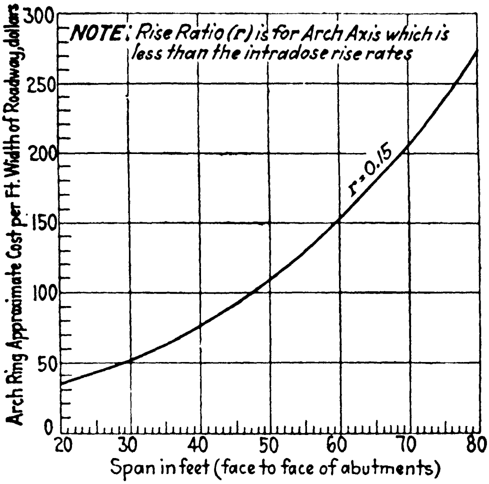
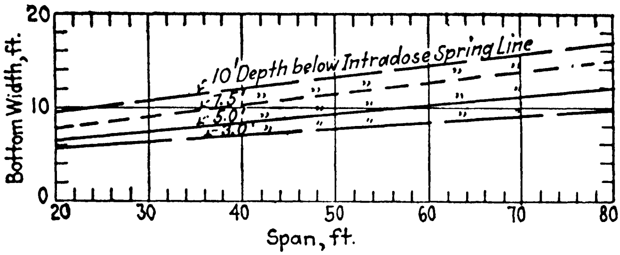


FIG. 100.—Approximate costs. Reinforced concrete arch rings. (Concrete $35 per c.y. incl. steel.)



Bottom Widths Abutments

NOTE: Curve marked 5' depth means bottom of foundation is 5' below intradose spring line

ABUTMENTS - Hard Pan or Rock Foundation

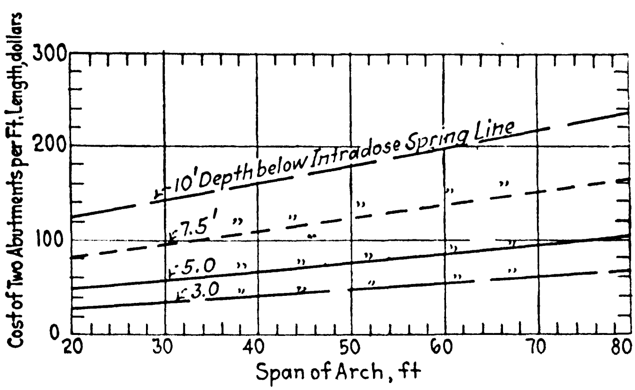


FIG. 101.—Approximate cost curves. Concrete arch. Bridge abutments. (Concrete at $20 per cubic yard.) Rise ratio of arch ring 0.15.

General Specifications, Reinforced-concrete Arches, Division 4

Rock or hardpan foundations:

Maximum pressure hardpan.....	6 tons per square foot.
Maximum pressure soft rock.....	8 tons per square foot.
Maximum pressure hard rock....	10–20 tons per square foot

Arches not recommended on pile foundations (12 tons maximum).
Arches not permitted on earth foundations
Arches not permitted with bottom ties (false arch) for spans over 10′.

Quantity and Cost Diagrams Based on Following Design Assumptions
(Trial Empirical)

Road grade 2.0′ above crown of arch, extrados crown thickness not less than one-sixtieth of span and varied to agree approximately with Schwada's formulas for highway arches (Ketchum, p. 417).

Spring thickness of arch ring:
3.0 × crown thickness for rise ratio of 0.125.
2.8 × crown thickness for rise ratio of 0.15.
2.5 × crown thickness for rise ratio of 0.20.
2.0 × crown thickness minimum for any condition.

Curve of trial arch axis based approximately on Cochrane's formulas for filled spandrel arches:

$$Y = \frac{4rL}{1 + 3r}(C^2 + 24\,C^6 r) \text{ (Hool and Johnston, p. 670).}$$

Minimum longitudinal arch reinforcement 1% at crown.
Transverse reinforcement to take full thrust of earth against spandrels and never less than 0.3% of section area at crown.
Arch reinforcement symmetrical.
Spandrel walls cantilever design up to 5 ft. height.
Gravity Spandrels or cross tie walls above 5 ft. height of fill.
Stirrups ¼″ circular spaced twice the depth of the arch ring at crown.

Unit Prices

Excavation as given on page 661.

Foundation and wing concrete..............	$20 per cubic yard.
Arch ring and spandrels......................	$35 per cubic yard.
Bar reinforcement..........................	6 cts. per pound.

Estimate graphs prepared for following parts of structure:
1. Arch ring 30 to 100′ spans, 0.10 to 0.25 rise ratio.
2. Wings.
3. Spandrels.
4. Foundations and abutments.
5. Piers.
6. Vertical-axis ordinates for Cochrane-formula-filled spandrels, different rise ratios (see p. 1086).
7. Table of arch axis angle with horizontal at spring line for different rise ratios (see p. 1086).

OUTLINE OF TYPICAL BRIDGE PRELIMINARY DESIGN REPORT

1. *Condensed recommendations:*
   - Location and skew angle of new bridge
   - Required waterway area
   - Channel improvements
   - Foundations, abutments, and piers
   - Superstructure:
     - Type
     - Span
     - Skew angle
     - Roadway
     - Sidewalks
     - Provisions for pipe lines or cables
     - Live load
     - Elevation of new structure
     - Camber
   - Temporary bridge for maintaining traffic
   - Incidental items
   - Estimated cost
2. *Detailed data and discussion.*
   - *a.* Location and skew angle of existing and proposed bridge
   - *b.* Waterway area of bridge:
     - Watershed
     - Probable, maximum run-off
     - Normal flood flow velocity
     - Ice and débris in stream
     - High- and low-water elevations
     - Waterway area of adjacent bridges on the same stream
   - *c.* Channel conditions and required improvements
   - *d.* Foundation conditions
   - *e.* Condition of existing bridge and other bridges on same stream (type and suitability)
   - *f.* Abutments, piers, and superstructure:
     - Type considering economic and esthetic requirements
     - Span (single or multiple)
     - Roadway width considering future traffic volume.
     - Sidewalks for pedestrians
     - Provision for pipe lines or cables
   - *g.* Temporary bridge for maintaining traffic
   - *h.* Incidental items:
     - Rip-rap
     - Approaches
   - *i.* Estimate of cost:
     - Source of materials
     - Alternate estimates different types, widths, and loads

FIELD INSPECTION AND PRELIMINARY DESIGN REPORT

Bridge 29, Livingston County
Bridge 5, Road 1247
Sta. 374 + 50A ±

Field Inspection: by W. G. Harger and R. W. Anderson
Preliminary Survey: by R. W. Anderson
Condensed Recommendation:
- New road approach relocation (see general layout, Fig. 48, p. 184)
- Right-angle structure recommended
- Required waterway below elevation 564.0 and above 538.0 = 5200 sq. ft. minimum.
- River channel to be improved
- Pile foundations
- Mass concrete abutments (see detail report)
- Superstructure:
  - Type—pin connected through steel truss
  - Clear span between abutments, 200′ minimum, 245′ maximum (see detail report)

Skew angle—no skew (right-angle structure)
Roadway—departmental minimum Class II road
Sidewalks—not required
Pipe line and conduits—none
Live load—H-15 or H-20 suitable
Elevation bottom of floor or truss steel 568.0 plus,
Elevation roadway surface
573.0 minus.
Camber—standard (approximate 12")
No temporary bridge
Abandon small bridges 4 and 6
Design of new approaches (see detail report)
Use concrete shoulders and rip-rap on slopes
Estimated Cost (rough preliminary; see detail report)

Road approaches (215' span bridge C to C bearings	$ 24,000
(255' span bridge C to C bearings	23,000
River channel improvement 215' span bridge	10,500
River channel improvement 255' span bridge	11,000
Bridge proper (see detail report):	
22' roadway (one high and one low abutment 255' span H-15)	72,000
22' roadway (two high abutments 215' span H-15)	80,000
30' roadway (two high abutments 215' span H-20)	105,000
30' roadway (one high and one low abutment 255' span H-20)	102,000
Total minimum cost (H-15-22' roadway 250' span)	105,000
Total maximum cost (H-20-30' roadway 215' span)	140,000

Detail Data and Discussion

Location and Skew Angle of Bridge.—The location of the existing bridge is poor both on account of sharp-approach road curves and poor flow conditions for the river, which retard the natural flow and cause ice jams and scour.

The attached general layout shows the proposed new location, which gives easy road alignment and a smooth flow for the Genesee River (see Fig. 48, p. 184). The new location also permits the use of a right-angle structure, which is desirable.

Required Waterway Area.—The Genesee River is subject to quick, severe floods with a large and rapid change in depth. It annually overflows its banks from Mt. Morris to Rochester and floods the entire valley for widths of from ¼ to 1½ miles. Road 1247 is often covered with water from 1.0 to 2.5' deep between Stas. 300 to 390, but these conditions rarely last over from 3 to 7 days and usually occur only once a year.

It has been customary to design all bridges over this part of the river to span the natural channel with the steel high enough to be above ice jams or floating débris (large trees), making no serious effort to raise the approaches above high water and to force all water under the main channel bridge.

Proposed water power storage dams, however, at Mt. Morris and Portageville, which may be constructed within 10 to 15 years, will tend to reduce the peak of the floods and it may easily become desirable to raise road No. 1247 above high water across the river flats to give uninterrupted year-around service to traffic.

It seems desirable to provide for both present and possible future conditions. Present conditions require a clear span of approximate 200 to 205' to span the normal channel at the crossing and an elevation of bottom of floor steel not lower than 568.0 to clear ice jams and floating trees.

Possible future flow conditions may be summarized as follows:

The Jones Bridge gaging station of the Rochester Gas and Electric Corporation located about 1½ miles upstream records normal floods for the last 16 years at 24,000 sec.-ft. approximately. In 1916 an extreme flood ran 46,000 sec.-ft. for 1 day and 55,000 sec.-ft. for a short period in that day. The effect of the proposed storage dams is, of course, problematical, but they will certainly reduce peak flows. According to the engineers of the Gas and Electric Corporation, a future peak flow of 30,000 to 35,000 sec.-ft. after completion of the Mt. Morris dam would probably be a very safe maximum flow allowance. It is recommended that future maximum flow under the bridge be based on 35,000 sec. at 7′ per second requiring a minimum flood-flow area of approximate 5000 sq. ft. This flood-flow area lies between Elevation 538.0 low water and 564.0 present high water, which it would be undesirable to raise due to damage claims and would require a clear span of approximately 200′. The previous discussion of present condition requirements indicated that a span of 200 to 205′ was required with a flood-flow area of 5300 sq. ft. between Elevations 538 and 564, which shows that any bridge satisfactory for present conditions should serve possible future conditions.

The present bridge on Road 1247 (No. 5) has a flow area of 5300 sq. ft., but 500 sq. ft. ± is scour area below Elevation 538.0, making the net effective flood-flow area 4800 sq. ft. The most recently constructed bridge over the Genesee River at Geneseo, where the watershed area is only slightly greater than at Road 1247, used a span of 222′ with a flood-flow waterway area of 5000 sq. ft. ±.

Spans of Genesee River bridges are as follows:

Bridge	Clear span, feet	Food-flow area under bridge, square feet
Bridge 29 (present)	180 + 130 = 310	5300
Bridge 29 (proposed)	200 to 205 minimum	5300
Jones bridge 1½ miles upstream	174 + 40 = 214	5000
Genesee, Road 718	222	
Avon, Road 5273	150	
Industry, Road 1393	235	
Ballantyne	215	
Browns Bridge, Road 1499	204	

A minimum clear span of 200 or 205′ and a minimum waterway area of 5200 sq. ft. between Elevations 538 and 564 should serve satisfactorily.

It will be well worth while to consider a maximum clear span of 240 to 245′ with the west abutment on top of the river bank. This will undoubtedly cheapen the bridge, as shown in the comparative estimates attached, and will probably serve satisfactorily for present conditions, as there is not much scour on the west side of the channel. Considering the future possibility, however, of forcing all the water under the bridge at a higher velocity, it is possibly safer to use two high abutments and the shorter spans, although I personally favor the cheapest first cost, as I do not consider there is much probability of scour and if it occurs it can be stopped at a small cost.

Channel Conditions.—Present channel conditions poor (see general layout and photos). Current velocity is needlessly checked; ice jams form easily and bad scour occurs on east bank. The material in the banks is river silt underlaid with clay. It scours easily.

The solution shown on the general layout seems obvious and not susceptible to much variation. Straighten channel, rip-rap east bank upstream heavily for at least 150′ above the bridge and for short distance below the bridge. Rip-rap new bank on west side for at least 80′ north and south of bridge. Use channel excavation for approach road fills. As a matter of fact, the channel excavation adds nothing to the total cost of this project, as borrow for the road approaches would have to be made in any case.

Foundation Conditions.—Soil river silt and clay; piles required. The only doubtful point is length of piles. It is better to pay the contractor for five or six test piles to determine this item than to waste money at this time with a well-drilling outfit to test for possible underlying hardpan, as it is known from scour channel of river that no hard strata are high enough to be used directly as foundation for masonry. For piles under abutment on top of bank above low-water elevation use concrete or creosoted piles.

Condition of Existing Structure.—Poor. Built in 1866. Safe load, 3 tons minus. Roadway 15.0′. Spans 180 + 130 = 310′ total. Abutments and piers excellent masonry. Reuse as plums in abutment concrete or as rip-rap.

Types of Bridges over Genesee River.—All bridges from Mt. Morris to Rochester are steel-truss bridges and only two of these bridges have piers;

these piers are not central piers and act in conjunction with short-approach spans. Central piers are not considered desirable on this river.

Abutments, Piers and Superstructure.—Piers not suitable or economical.

East abutment should be a high abutment with pile foundations and face sheeting to prevent undercut scour. Mass concrete with 20% plums utilizing old masonry is advised, as this abutment will have to take considerable punishment from ice, etc. Elevation bottom concrete 535.0 minus, which is 4′ below surface of low water.

West abutment can be either high abutment at bottom of bank or low abutment on top of bank. Piles will be required for either location.

Best type of superstructure probably steel through truss parabolic upper chord pin connected solid floor similar to recently constructed bridges at Industry, Ballantyne, and Geneseo.

Design loading and roadway width should be carefully considered, as they result in considerable difference in cost for this structure.

This road carried 668 vehicles 1925 traffic census. Character of traffic largely pleasure vehicles (light motors). This is a secondary route and there is no possibility of it ever exceeding 3000 vehicles daily which can be served by a two-lane bridge. No provision for pedestrian traffic is necessary.

The A.S.C.E. recommend H-15 loading for these conditions and a minimum 20′ roadway. The Hoover Committee on Highway Safety recommends a minimum roadway of 22′ on account of busses for these conditions. The departmental standards use H-20 and 30′ minimum roadway which, while they are desirable on really heavy-traffic roads, cannot be considered as necessary for this particular case.

Temporary Bridge.—Not feasible.

Road Approaches.—Concrete pavement with concrete and rip-rapped shoulders similar to Road 1393. 4.2% maximum grade. Elevation of grade across flat 562.5, which is the same as present approaches and will not increase ponding on upstream side of road with consequent claims.

Abandon small bridges 4 and 6.

ROUGH COMPARATIVE COST ESTIMATES BRIDGE 29, LIVINGSTON COUNTY; BRIDGE 5, ROAD 1247

Suprestructure:

215′ span C to C of bearings H-20-22′ roadway	$35,000
215′ span C to C of bearings H-20-30′ roadway	48,000
215′ span C to C of bearings H-15-22′ roadway	28,000
215′ span C to C of bearings H-15-30′ roadway	39,000
255′ span C to C of bearings H-20-22′ roadway	45,000
255′ span C to C of bearings H-20-30′ roadway	62,000
255′ span C to C of bearings H-15-22′ roadway	36,000
255′ span C to C of bearings H-15-30′ roadway	50,000

ABUTMENTS (STEEL AND MASONRY ONLY)

($14 per cubic yard and $0.06 per lb. 20% plums old masonry)

Two high abutments (22′ roadway)	$35,000
Two high abutments (30′ roadway)	39,000
One high and 1 low (22′ roadway)	21,000
One high and 1 low (30′ roadway)	24,000

ABUTMENT PILES AND SHEETING IN PLACE

($1 per foot, $80 per M ft B.M.)

30′ roadway

Two high abutments	$10,000 + 1,000 = $11,000
One high and one low	7,000 + 1,000 = 8,000

ABUTMENT EXCAVATION AND BACKFILL

(Below Elevation 540 and back of face of abutments)

Two high abutments	$8,000
One high and one low	6,000

ROUGH ESTIMATE ROAD APPROACHES

(Same general design as Industry River Flat Road 1393 concrete shoulders, down to natural surface of ground on light fills and rip-rap on higher fills. Borrow fill to be obtained from river channel widening.)

	215′ span	255′ span
Pavement (concrete 18′ wide) at $3.20 per sq. yd.	$10,300	$10,000
9″ concrete shoulders (reinforced) at $3.00. per sq. yd.	5,400	5,400
400 cu. yd. ± rip-rap at $4 cu. yd.?[a]	1,600	1,[illegible]00
Guard rail at $1.50 per lin. ft.	1,800	1,700
Excavation 500 at $1	500	500
10,000 cu. yd. ± borrow fill (extra cost for placing channel excavation at 40 cts.)	4,000	3,500
Clearing and grubbing L. S.	500	500
Total	$24,100	$23,000

[a] Price of rip-rap not figured closely.

Rough Estimate Channel Improvement

	215′ span	255′ span
600 cu yd. ± excavating old masonry at $4 [b].	$ 2,400	$ 2,400
500 cu. yd. ± rip-rap 500 / 700 at $4?[b]	2,000	2,500
10,000 cu. yd. ± earth excavation at 60 cts.	6,000	6,000
Clearing and grubbing, L. S.	300	300
Total	$10,700	$11,200

[b] Old masonry used for rip-rap or for plums in abutment concrete.

Note.—Channel excavation to be used in road approach fills or, if wasted, contractor to furnish other borrow at his own expense.

Photographs and detail study profiles and general layout attached.

Signed
W. G. Harger,
Bridge Eng. Div. No. 4
Oct. 31, 1925

FIELD INSPECTION AND DESIGN REPORT BRIDGE 25, LIVINGSTON COUNTY BRIDGE 1, ROAD 1247 STA. 248, BEARDS CREEK

Field Inspection: by W. G. Harger and R. W. Anderson, Oct. 20, and 21, 1925.

Condensed Recommendations:

Location and skew angle (see general layout):
- New road approach center line.
- Intersection center lines bridge and New Creek Channel Sta. 247 + 62 ±.
- Skew angle 48°.

Required waterway area: 500 sq. ft. minimum.
540 sq. ft. recommended.
- Measured at right angles to stream flow and between Elevations 576.0 and 585.0.

Channel improvements (see general layout):
- New outlet channel 200′ long.
- **Cut off small bend in east bank of inlet channel—250′ above bridge to straighten channel.**

Foundations abutments and piers:
- **Short piles down to hardpan (piles 15′ long ±).**
- **Standard concrete abutments.**
- **Piers are unsuitable for this bridge.**
- **Elevation of bottom of foundation concrete 571.5.**
- **Foundation excavation will have to be sheeted and pumped.**

Superstructure:
Type steel-plate girder (concrete floor).
Span 85' clear between faces of abutments 90' ± C. to C. of bearings.
Skew angle 48°.
Roadway departmental minimum for Class II road.
Sidewalks—none required.
Pipe lines or conduits—none.
Live load—H 15 or H 20 suitable.
Elevation bottom of steel 586.5+.
Elevation of roadway surface 590.5−.
Camber—standard camber.

Temporary bridge:
Should be provided if this bridge is let by a separate contract. If let as part of road reconstruction no temporary bridge needed.
Good detour 1 mile extra distance.

Incidental items:
Rip-rap approach fills 100' each side of bridge on north side of road and 60' each side of south side of road.
Future road approach grade across valley 587.0.

Estimated Cost:
Estimated cost of new bridge and channel work exclusive of new approaches as follows:

22' roadway (minimum	$31,000
30' roadway (maximum)	38,000

Detail Data and Discussion

a. **Location and Skew Angle of Existing and Proposed Bridge.**—The accompanying general layout and the testimony of field indications and local residents show that the existing bridge is poorly located and causes scour and needless checking of flow and ice jams. The new bridge should be moved west about 30 to 35' and should be constructed on a skew angle of approximately 48°.

b. **Waterway Area Required under Bridge.**—Watershed 30 sq. miles classed as midway between rolling and hilly country.

Probable maximum flood 3500 sec.-ft.

Probable flood velocity of flow, using actual stream channel cross-section and slope of stream 6 to 8' per second based on Church's diagrams, Kutter's formula.

Flow area under bridge based on 3500 sec.-ft. at 7' per second should be safe—500 sq. ft. minimum.

Waterway area of other bridges on stream are not of much value, as conditions of flow are entirely different. They are tabulated as follows and indicate in a general way that it is desirable to use about 540 sq. ft. under this bridge.

WATERWAY AREA OF ADJACENT BRIDGES OVER BEARDS CREEK

Location	Span		Waterway area, effective at right angles to stream flow, square feet	Road approaches, overflow area, square feet	Total area, square feet
	Skew feet	Effective right angle feet			
Bridge number 25	86'	55 ±	460 ±	Indefinite	Indefinite
Combined four bridges of Beards Creek branches upstream	..	84	470	....	480
First bridge downstream[a]	..	41	300	....	300
Second bridge downstream Pennsylvania Ry.[a]	..	50	350	Indefinite	Indefinite
Third bridge downstream, road 1247[b].	..	40	540'	Indefinite	Indefinite

[a] These bridges are admittedly too small.
[b] Back water from river reduces value of these data.

c. **Channel Conditions.**—Slope of stream, 0.25%. Present channel very crooked at bridge site and needs straightening (see general layout). Large amount of ice which jams due to crooked channel. Proposed channel straightening shown on general layout approximate 900 cu. yd. excavation. It is impracticable to lower channel under bridge below elevation 576.0.

d. **Foundation Conditions.**—Light soil underlaid with hardpan or shale at elevation 562 ±. Present abutments on short piles down to hardpan. Short blunt end piles down to hard pan recommended figured for 15 tons per pile.

e. **Condition of Existing Bridge 25 and Other Adjacent Bridges over Beards Creek.** *Bridge* 25.—Concrete abutments in fair condition, 86′ clear span, steel pony truss plank floor. 15′ roadway. Steel in poor condition. Safe load 4 tons figured by W. G. Harger.

Bridge 25 should be condemned as soon as funds become available for rebuilding.

Six bridges over this creek are concrete-slab structures and two are single-span steel trusses or girders. One bridge has a central pier and has not been satisfactory due to scour and ice jams. Central piers are apparently not desirable on this stream.

f. **Abutments, Piers, and Superstructure.**—Piers are undesirable on this stream.

For a single-span bridge on pile foundations, the plate girder type is economical, and for this location appearance has no weight in the decision. A plate girder with solid floor is recommended.

Considering the facts that it is impracticable to lower the stream bed and undesirable to raise high-water elevation on account of flooding road across flat, a span of 57′ at right angles to stream flow is recommended, which requires 85.0′ clear span between faces of abutments measured parallel with proposed new road center line. This will give a waterway area of 513 sq. ft. measured at right angles to stream flow below Elevation 585.0, which is present high water.

Road 1247 is a Class II highway which carried 668 vehicles according to traffic census of 1925. Making ample allowance for future growth, this road will be amply served with a two-lane bridge requiring a minimum roadway of 22′.

Sidewalks are not required nor is any provision necessary for pipe lines, conduits, or electric trolley tracks.

Considerable heavy-unit trucks use this road, which warrants an H-20 loading, although H-15 would probably serve satisfactorily.

Bottom of steel girders or floor of superstructure should be at Elevation 586.5 or higher.

Road grade 590.5 or as much lower as possible so long as the bottom of steel is not less than 586.5.

g. **Temporary Bridge.**—Good detour 1 mile extra distance.

Temporary bridge recommended in case this bridge is let as a separate contract. No necessity in case it is included in a reconstruction road contract.

h. **Incidental Items.**—Approach fills to be rip-rapped. Approach alignment changed to eliminate reverse curve east of bridge and to ease up west approach. New tangent advised from Sta. 244 of present road to 253 + 50 ± with easy curves at 244 and 253.

i. **Estimate of Cost.**—Rough appropriation estimates of cost are as follows, using H-20 loading:

	Minimum, 22′ roadway	Maximum, 30′ roadway
Superstructure	$13,500	$18,300
Abutments and piles	13,300	15,000
Excavation	2,500	3,000
Channel cleaning	700	700
Temporary bridge	1,500	1,500
Right of way	500	500
Totals	$32,000	$39,000

Signed
Bridge Engr.
Div. No. 4

LONGITUDINAL DRAINAGE

Longitudinal drainage covers the normal road-section ditch, special creek channels, the protection of ditches from scour on steep grades, the use of storm water sewers on long grades where it is not possible to get rid of the surface water by diversion from the road, and driveway culverts.

Carrying Capacity of Ditches.—It is desirable to use as shallow and small a road ditch as possible on the score of both safety and economy of grading (see sections p. 143). All ditches will clog more or less in the winter with snow and ice, so that the size of the ordinary road ditch is more a matter of judgment based on experience than it is of figures. Intercepting ditches carrying the run-off from considerable areas should be figured, using run-offs similar to culvert design methods.

Creek channels and intercepting ditch capacities can be figured. Church's diagrams of Kutter's formulas using a value of $N = 0.035$ furnish a quick, easy method of approximately the required size (see Table 52, p. 195).

In order to give a fairly definite idea of the limitations of the use of the shallow and medium road ditches shown in Fig. 102, the following conditions are outlined. As a rule, the capacity of the roadway ditch is taxed by short, sharp summer showers. As

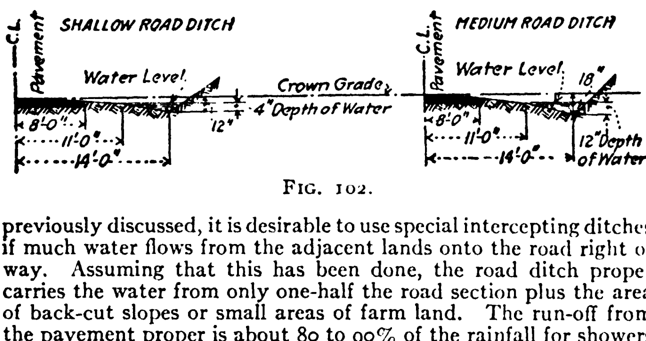


FIG. 102.

previously discussed, it is desirable to use special intercepting ditches if much water flows from the adjacent lands onto the road right of way. Assuming that this has been done, the road ditch proper carries the water from only one-half the road section plus the area of back-cut slopes or small areas of farm land. The run-off from the pavement proper is about 80 to 90% of the rainfall for showers of, say 10-min. duration. The run-off from the shoulders and backslopes is perhaps 60% under favorable conditions. An average run-off of about 75 to 80% of the rainfall for the area of one-half the total right-of-way width can be assumed. This means, as a rule, that the shallow ditch should not be used for more than 400′ from a summit or below a ditch relief culvert on flat grades nor more than 800′ on moderate grades. Current practice recognizes this general principle by the use of deeper and larger ditches in flat country than in rolling country. The medium ditch will serve satisfactorily for at least 2000′ from summits, provided it does not carry side land run-off.

To give a rough idea of the carrying capacity of the ditches condensed Table 59 is given for a few ordinary cases.

TABLE 59.—APPROXIMATE CARRYING CAPACITIES AND VELOCITIES OF FLOW, ORDINARY ROAD DITCHES GRASSED OVER

Grade of road ditch, per cent	Shallow ditch (Fig. 102)		Medium ditch (Fig. 102)			
			Depth water, 12″		Depth water, 6″	
	Velocity, feet	Capacity second-feet	Velocity, feet	Capacity second-feet	Velocity, feet	Capacity second-feet
1.0	0.7	0.4	2.0	4.0	1.1	0.5
2.0	1.1	0.7	3.0	6.0	1.7	0.8
3.0	1.3	0.8	3.6	7.2	2.0	1 0
4.0	1.5	0.9	4.2	8.4	2.3	1.2
5 0	1.7	1.0	4.8	9.6	2.6	1.3
6.0	1.8	1.1	5.3	10.6	2.8	1.4
7.0	2.0	1.2	5.6	11.2	3.1	1.5
8.0	2.2	1.3	6.0	12.0	3.4	1.7
9.0	2.3	1.4	6.5	13.0	3.6	1.8
10.0	2.5	1.5	7.0	14.0	3.8	1.9

Protection of Ditches from Scour.—The rate of grade at which ditch protection from scour is advisable depends on the soil and velocity of flow; the velocity of flow depends on the shape of ditch and volume of water.

Soils scour at approximately the following velocities:

Sand	2–3 ′ per second
Loam	2–3½′ per second
Firm gravel	5–6 ′ per second

Table 59 is inserted to show in a general way the effect of shape of ditch and depth of flow on velocity and indicates that ditch protection must be provided at lower rates of grade for a large than for a small flow. This agrees with current practice, which favors the use of cobble or cement gutter at approximately the following rates of grade.

Current Practice in Ditch Protection.—Where the volume of flow is less than 1.0 sec.-ft., ditch protection is not needed on hills less

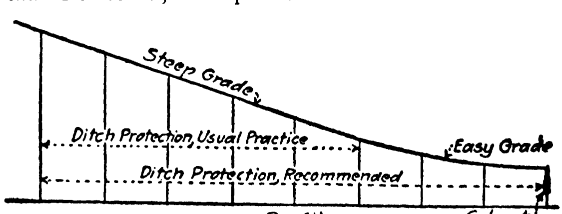


FIG. 103.

than 7% grades. Where the volume of flow exceeds this amount, ditch protection is advisable on sandy or loam soils on grades steeper than 3% and in firm gravel on grades of 5% or greater.

This means that, as a rule, on steep grades some kind of gutter is desirable for ditches more than 200 to 300′ from a summit or below a ditch relief culvert. It is practically impossible to carry a large volume of water down a steep grade, so that every effort should be made to divert the flow above the grade or remove it from the surface by ditch relief culverts or storm sewers.

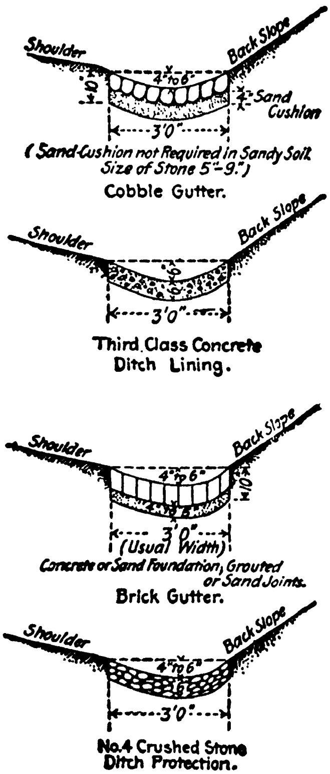


FIG. 104.

Where ditch protection is used it is good practice to carry it for at least 200′ along the road after the foot of the steep grade is reached, and preferably to the first culvert below the grade in question, as scour often occurs through stopping the protection too closely to the bottom of the steep grade.

Cobble gutter with cement joints on grades over 6% and sand or gravel joint filler on grades less than 6% where the volume of flow is not large make probably the best design, as they tend to retard the velocity of flow.

The smooth concrete ditch lining is not usually satisfactory on steep grades but is allowable if the cobble gutter is not available.

Storm Sewers on Hills.—Where it is impossible to divert the water from the surface on long hills a storm-sewer system is sometimes used.

Catch-basin inlets are constructed at intervals of 200 to 400′.

The sizes of pipe are figured for probable run-off in the same manner as for culverts using formulae from Table 46, page 188 with a coefficient *c* of approximately 0.6. Table 60 gives a rough approximation of the carrying capacity of different sized pipes laid on different grades.

TABLE 60.[1]—APPROXIMATE FLOW CAPACITY IN CUBIC FEET PER SECOND

Value of $N = 0.013$

Grade, per cent	Capacity of flow of different sized pipes					
	12″	15″	18″	20″	24″	36″
0.5	2.4	4.4	7.5	9.5	16.0	42.0
1.0	3.3	6.3	10.5	14.0	23.0	60.0
1.5	4.2	7.6	13.0	17.0	27.0	75.0
2.0	4.8	8.8	15.0	19.0	31.0	86.0
3.0	5.8	11.0	18.0	24.0	39.0	105.0
4.0	6.5	13.0	22.0	27.0	46.0	122.0
5.0	7.3	14.0	24.0	30.0	51.0	137.0
6.0	8.1	15.0	26.0	33.0	56.0	150.0
7.0	8.8	16.0	27.0	35.0	60.0	162.0
8.0	9.5	17.0	28.0	38.0	65.0	173.0

[1] Computed from diagram in Ogden's "Sewer Design."

Figures 105 and 105*A* give typical catch-basin and simple manhole designs for a sewer.

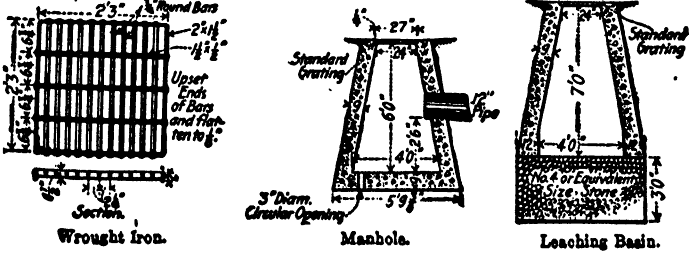


FIG. 105.—Drop inlets and catch basins.

Minimum permissible storm-sewer grades to insure against clogging are given in Table 61.

TABLE 61.—MINIMUM GRADES FOR STORM SEWERS TO PREVENT OBJECTIONABLE CLOGGING

Diameter of pipe, inches	Minimum permissible grade, per cent[a]
12	0.34
15	0.24
18	0.18
24	0.12
30	0.09
36	0.07
42	0.055
48	0.045

[a] Velocity of flow, half full = 2.5′ per second.

Driveway Culverts.—Culverts under drives cause more drainage troubles than any other feature, as they are usually constructed by the property owner instead of being included in the road contract.

It is an amusing fact that private farm bridges over streams or road ditches are notoriously small. Each owner seems to think that water will get under his drive in a small, cheap structure, and the same man who says that a road structure is too small will build a bridge or driveway sluice just below this structure that he has been kicking about and make it from one-tenth to one-half as large as the main drainage structure unless he is prevented by law which is enforced by highway officials.

The size of the structures should be fixed on the road plan and designed from the same standpoint as a culvert or bridge.

Cheaper types of structure are suitable, as they can be readily replaced without tearing up the pavement.

Corrugated metal pipe, vitrified tile, or reinforced-concrete tile are suitable, but no matter how little water is carried the size of driveway culvert should not be less than a 12″ pipe on account of maintenance difficulties. These culverts are properly placed in the ditch line at normal ditch grade and, as a rule, 12′ length is about the minimum that will be at all satisfactory.

UNDERDRAINAGE

The purpose of underdrains on hard-surfaced roads is to intercept the ground water before it reaches and softens the subgrade. On a side-hill road the drain is usually placed under the ditch on the uphill side (see Fig. 106, position 1), where the greatest depth can be obtained with the least excavation and where the water is caught as it flows out of the hill.

Some engineers place the drain in position 2 (Fig. 106), but this requires more excavation for the same depth and for side seepage is not so effective. The usual depth for drains is 3 to 4 ft. below the surface.

Where the road is on a descending grade, the water will flow out of the hill directly under the stone and the drain is placed as in Fig. 107,

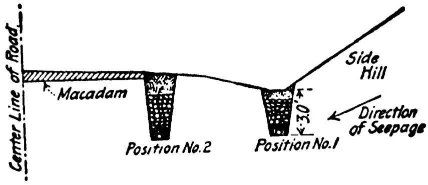


FIG. 106.

position 1, or two drains are built in position 2. Position 1 is the usual practice, being cheaper and more effective.

Two side drains are effective. In case the throat becomes clogged, a side drain can be taken up without disturbing the mac-

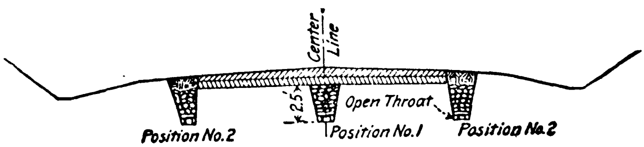


FIG. 107.

adam. This rarely occurs in a center drain, as it is better protected than those in position 2 and in case the center drain does clog, side drains can be constructed at any time.

There are two kinds of drain in general use (Fig. 108).

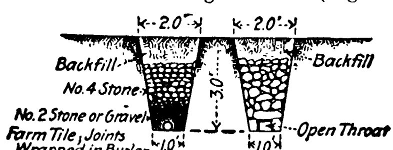


FIG. 108.

No. 1 is built entirely of stone with an open throat roughly laid as shown; it is satisfactory in water-bearing strata of gravelly loam or clay, but does not work so well in quicksand, which is liable to fill it up. It is generally cheaper, however, than No. 2.

No. 2 is built of porous farm tile or vitrified tile of a suitable size (usually 3 to 6″) with open joints, wrapped with a double or triple layer of burlap; the pipe is surrounded and covered with clean gravel or ¾″ crushed stone to a depth of 6″, the remaining depth of the trench being filled with large stone. If this drain has a good fall and the outlet is kept free, it will rarely clog even in bad quicksand.

The following method has been successfully used to prevent the outlet from clogging; after being brought out from under the macadam, the drain is continued under and across the ditch line; then, keeping outside the ditch line, and using a slightly smaller gradient than that of the open ditch, the tile is continued down the hill until it reaches a point 8 or 9″ below the ditch grade. Here it is turned into the open ditch through a small concrete headwall and what little

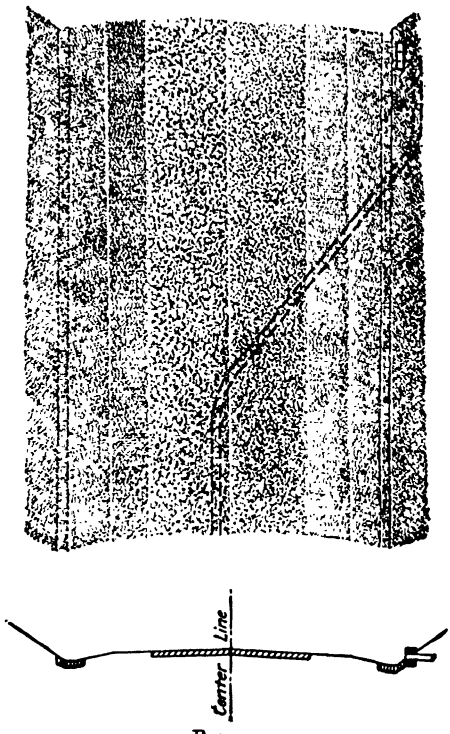


FIG. 109.

material it tends to deposit is washed down the ditch by the surface water (see Fig. 109). The lowest rate of grade advisable for underdrains is 3″ per 100′.

Summary of Chapter.—The present bridge situation demands attention, as even in the richer states it is often lagging behind the improvement of the roads. The separation of bridge and highway funds and the lack of central control often result in the ridiculous situation of a modern road limited in use by antiquated bridges.

Road pavements can be strengthened from year to year by increasing their thickness and by the construction of better surfaces on top of existing improvements, but structures must be rebuilt entire to increase their strength, and for this reason more foresight in regard to future traffic must be exercised in their design. A liberal allowance for increased loads is desirable. Liberality in size of waterway for culverts is also good policy, as it adds only slightly to the cost and materially decreases the difficulties of maintenance.

The design of drainage must be complete and reasonable, and if the existing scheme is not feasible it should be changed regardless of lawsuits as, whenever an improvement is made, it is always cheaper to correct mistakes at that time than it will be at a later date, as every year's use fixes the channels more firmly.

The selection of type and utilization of parts of existing structures offer the greatest chances for reasonable economy in culvert and bridge design.

CHAPTER V

EARTH, SAND-CLAY, GRAVEL, AND MISCELLANEOUS LOW-TYPE ROADS

Introduction.—Roads of the type discussed in this chapter form the groundwork of future high-class pavements and represent the greater percentage of mileage of roads in this country see page 42. They are entitled to more engineering supervision than they have received in the past; that is, they should be improved by a series of well-thought-out related steps of progressive improvement (page 2).

These types of construction are the initial steps in final road. improvements and serve gradually to pull traffic "out of the mud." They are the only types which can be reasonably built in sparsely settled pioneer districts or agricultural districts of low assessed valuation. Earth and sand-clay roads can, however, be classed only as temporary makeshifts under adverse weather conditions (5 months in the year) on account of softening in wet weather. Gravel roads serve very well as a final stage of improvement for local roads carrying light traffic, provided they are made thick enough to handle 2½-ton trucks the year round. Maximum traffic volumes for the gravel type are discussed on page 353 and for all types of road have been indicated in a broad general way in Chap. I (p. 6).

The following table from Agg's "Construction of Roads and Pavements" reinforces the discussion of traffic limits for the low-type roads. It can be readily seen that these types are suitable for a large percentage of the road improvements in the United States (1926).

TABLE 62.—AVERAGE DAILY TRAFFIC LIMITS IN MASSACHUSETTS

Table showing results of observations of traffic on different types of road surfaces in Massachusetts. Standard road, 15 ft. in width; gravel or water-bound macadam, 5 or 6 in. in thickness, with adequate drainage and proper foundation, with 3-ft. gravel shoulder on each side.

Type of Surface	Light Teams, Carriages, Wagons	Heavy Teams, One-horse	Heavy Teams, two or more Horses	Automobiles
A good gravel road will wear reasonably well and be economical with..........	50–75	25–30	10–15	50 to 75
Needs to be oiled with....	50–75	25–30	10–15	Over 75
Oiled gravel, fairly good, heavy cold oil, ½ gal. to the sq. yd. applied annually with....................	75–100	30–50	20	500 to 700 or more
Waterbound macadam will stand with...............	175–200	175–200	60–80	Not over 50 at high speed
Cold oil or tar will prove serviceable on such macadam with................	175–200	175–200	60–80	50–500
Macadam will then stand, but the stone wears, of course, with.............	175–200	175–200	60–80	500 or more
*Waterbound macadam with hot asphaltic oil blanket will be economical with....	100–150	50–75	25–30	1500 and more with fewer teams
And stand at least.........	....	....	....	50 trucks
But will crumble and perhaps fail with over....... (On narrow tires, ice, farm and wood teams, etc.)	150	75	30	
*Waterbound macadam with a good surface coating of tar (½ gal. to the sq. yd.) will stand with........... (But requires to be recoated annually with ¼ gal. of tar per sq. yd.)	100–150	50–75	25–30	1500 or more

It is assumed that all road surfaces are kept constantly patched, that before applying bitumen the road surface is cleaned and patched, and the bitumen covered with pea stone and sand or gravel and kept covered so that it never picks up.

* AUTHORS' NOTE.—One coat penetration bituminous macadam will stand any number of light autos and more steel tire or truck traffic than shown above, because it takes the wear more directly and has no blanket coat which crumbles under such traffic.

EARTH ROADS

Rut Roads.—The simplest form of road is the so-called rut road used in the arid regions of New Mexico and the Southwest. They are constructed by clearing the right of way of brush and then cutting two shallow parallel ruts in the surface vegetation or soil crust by means of two cutting irons gaged to fit the ordinary wagon track. A wagon trail of this kind can be constructed for $10 to $30 per mile, can be used by autos with fair comfort at speeds up to 15 m.p.h., and on the flat mesas of this district are more lasting and satisfactory than the ordinary turnpiked section, as so little rain falls that an elevated fill grade does not consolidate and is

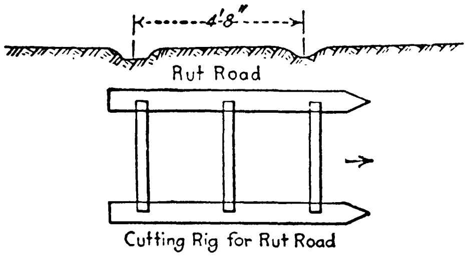


FIG. 110.—"Rut roads."

worse than useless for traffic. On these rut roads any rain storms that occur wash the coarser particles of the soil into the ruts and gradually an armored track is formed below the general elevation of the mesa. No drainage structures are necessary where construction of this type is adopted.

Earth Roads.—The same principles of grade, section, and drainage apply to this class of road as to the higher types except that the surface ditches are generally made slightly deeper and more care is taken with the underdrainage; this is necessary as the earth road becomes more easily saturated with water than types which are sealed over on the surface. If the natural soil is good road material, such as gravel, disintegrated rock, hardpan, or sandy loam, this type of construction carefully graded, drained, and shaped by blader finish and maintained by dragging makes a satisfactory road for light traffic. Their cost depends on the amount of grading required and the methods that can be used. The cost of drainage culverts, incidentals, etc. will vary, but will run about $600 per mile for good work.

Simple blade-machine turnpiking, where the dirt from the ditches made the center fill, cost (in districts similar to Wyoming 1914–1915) about $150 per mile. The same work at present (1918) is bid off for about $200 per mile. A fair relative price for first-class work of this kind, including drainage and incidentals, can be placed at $600 to $800 per mile.

In rolling country requiring grade reductions by cut and fill and wagon haul, a fair relative price, including drainage and incidentals, is approximately $2000 to $4000, where no rock is encountered.

In mountain-road work where the excavation runs anywhere from 1000 to 30,000 cu. yd. per mile with a large percentage of rock the

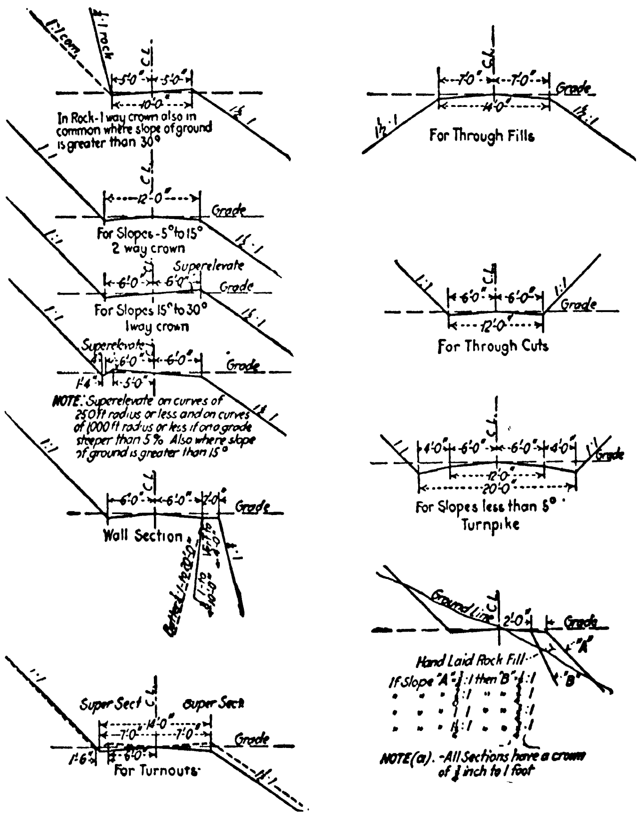


FIG. 111.—U. S. Bureau of Public Roads standard mountain earth road sections.

cost will run anywhere from $1000 to $25,000 per mile. A fair average for such conditions is $5000 to $8000 per mile (1922 cost conditions).

As previously stated, it is entirely a matter of required grading.

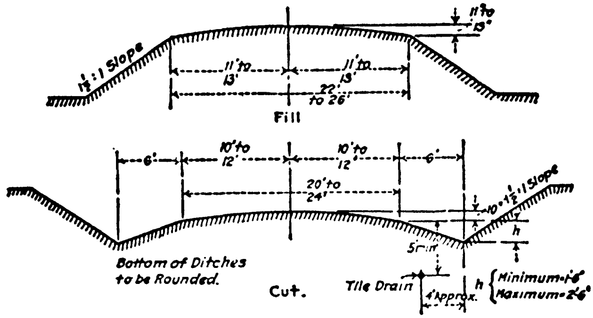


FIG. 112.—Iowa earth road sections.

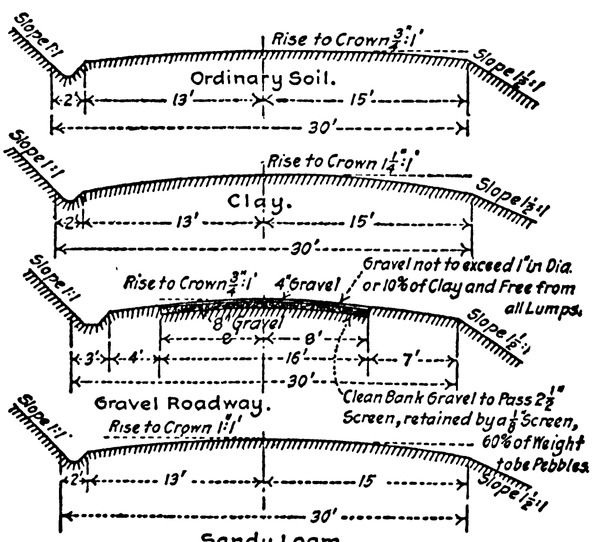


FIG. 113.—Typical earth and light gravel road sections, State of Pennsylvania.

Current practice in earth-road sections is shown in the following figures and also in Chap. III (pp. 146 to 171).

Mountain roads............................ Fig. 111 (p. 345).
Iowa standards............................ Fig. 112 (p. 346).
Pennsylvania standards...................... Fig. 113 (p. 346).

Current practice in grading and finishing is given in the typical specifications (p. 1444).

Earth-road maintenance is discussed in Chap. VII. The cost ranges from $20 to $200 per mile per year and for ordinary conditions probably averages about $50 to $100. Where it rises above these amounts a better grade of surface is g nerally constructed.

Where the soil is not a good road material the surface is improved, by artificial mixtures of selected soil or by surfacing with gravel, chert, disintegrated granite, slag, shell cinders etc.—in fact, any local material that gives body to the surface and prevents softening during the wet season.

SAND-CLAY ROADS

A poor earth road can be very materially improved by the so-called sand-clay surfacings. A well-selected proportioning of sand and clay increases the resistance of a poor earth road both to traffic wear and to softening due to moisture. A sand-clay surface cannot, of course, be considered as a satisfactory all-year-round road for heavy load, but it is the easiest and cheapest means of increasing the ease and comfort of travel for a large part of the year, and this type of road has had considerable popular approval, particularly in the southern states. It is economically justified for light volume of traffic in financially poor districts where good road gravel is not available. Sand-clay construction is not advised in the northern states where gravel is available.

Typical sand-clay crowns, widths, and thicknesses are shown in Fig. 114 and in Chap. III (pp. 147 and 148).

The cost of surfacing with sand clay varies, as any form of construction, with labor, length of haul, cost of materials, etc., but generally adds from 15 to 35 cts. per square yard to the cost of an earth road. A fair comparative figure would add $1500 to $2500 per mile for a 16′ width over earth roads and costs about the same as earth-road maintenance. Specifications for sand clay are given on page 351.

Proportioning of Soil Mixture.—Where the natural soil is clay the resisting power of the surface during wet weather can be increased by the addition of sand. Where the natural soil is deep sand the surface can be made firm and resilient by the addition of clay. The so-called sand-clay treatment aims to provide a surface layer of mixed sand and clay about 10 to 12″ deep (see Fig. 114, Alabama Standards), in which the sand forms the body and the clay just fills the voids in the sand and acts as a binder. It can be readily seen that different materials will require different proportioning of the sand and clay. The only sure way to get the best results is by experiment on the road during construction, but to give an idea of the approximate proportioning the following list of recommended mixes is taken from the "Good Roads Year Book" of the American Highway Association, 1917.

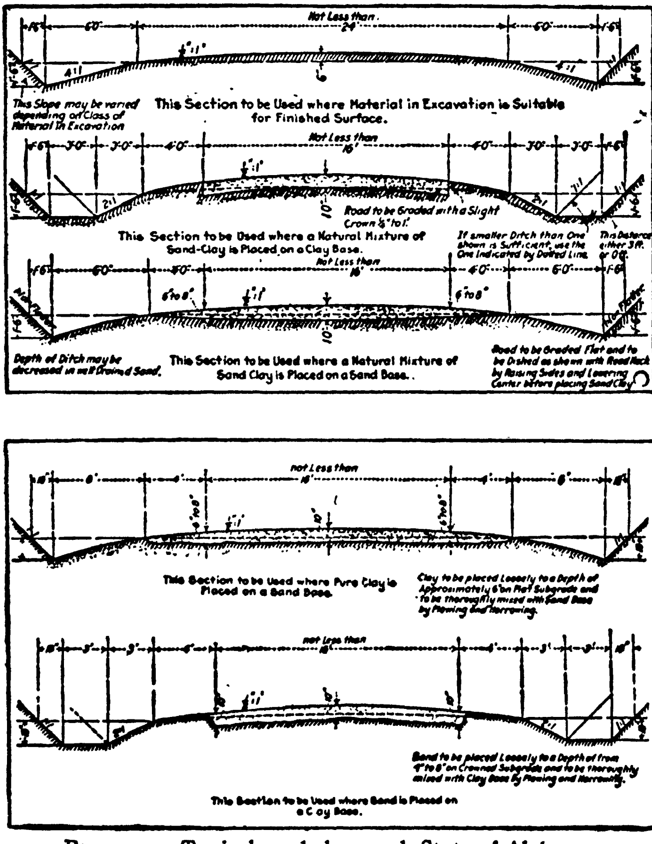


FIG. 114.—Typical sand clay road, State of Alabama.

"The grains of which sand is composed are usually hard and tough and able to resist abrasion if held securely in place. In an asphalt pavement they are held by the asphalt and a wearing surface of great resistance to abrasion results. In a sand-clay road they are bound together by clay in a less firm manner but one giving excellent results on well-drained roads carrying light traffic. The aim of the builder of such a road is to employ just enough of the stickiest clay at his command to fill the pores of the sand and to mix these materials together so thoroughly that there are neither lumps of clay nor pockets of loose sand left in the surfacing. This gives the maximum amount of hard sand to carry the traffic and the minimum amount of clay to bind it. More sand makes a less durable road and more clay makes one which becomes soft more rapidly when wet.

"There is a great difference in the value of different clays for such work. Some of them become dough-like when mixed with a certain amount of water and can be molded into objects which retain their shape after drying. If these molded objects are immersed in water they will retain their form for a long time. These varieties are called 'plastic clays' and the most plastic are called 'ball clays.' Other varieties fall to pieces more or less quickly when wet, as quicklime does, and they are therefore called 'slaking clays.' They are more easily mixed with sand than the plastic clays, but they have much less binding power and a road built with them is less durable when dry and more easily rutted when wet. The amount of clay to be used can be determined by a simple field test described as follows by Andrew P. Anderson:

"'From typical samples of each of the available clays, test mixtures, varying by one-half part, are made with the sand so that each clay is represented by a set of mixtures ranging by successive steps from one part sand and three parts clay to four parts sand and one part clay. These are worked up with water into a putty-like mass and from each mix two equal quantities are taken and rolled between the palms of the hands into reasonably true spheres, labeled, and placed in the sun to dry. When thoroughly baked, a set of spheres representing any one clay is placed in a flat pan or dish and enough water poured gently into the pan to cover them, care being taken not to pour the water directly on the samples. Some samples will begin to disintegrate immediately. Those breaking down most slowly contain most nearly the proper proportion of sand and clay for the particular materials. The relative binding power of the various clays may then be determined by comparing the hardness and resistance to abrasion of the various dry samples having the correct proportion of sand and clay, as determined by the water tests.'"

"In February, 1917, representatives of 21 state highway departments of the U. S. Office of Public Roads recommended the following mixtures for hard, medium, and soft classes of sand-clay roads.

"*Hard Class.*—Clay, 9 to 15%; silt, 5 to 15%; total sand, 65 to 80%; sand retained on a 60-mesh sieve, 45 to 60%.

"*Medium Class.*—Clay, 15 to 25%; silt, 10 to 20%; total sand, 60 to 70%; sand retained on a 60-mesh sieve, 30 to 45%.

"*Soft Class.*—Clay, 10 to 25%; silt, 10 to 20%; total sand, 55 to 80%; sand retained on a 60-mesh sieve, 15 to 30%.

"By clay is meant material separated by subsidence through water and possessing plastic or adhesive properties; it is generally below 0.01 mm. in diameter. By silt is meant the fine material other than clay which passes a 200-mesh sieve and is generally from 0.07 to 0.01 mm. in diameter. By sand is meant the hard material which passes a 10-mesh and is retained on a 200-mesh sieve, and is generally from 1.85 to 0.07 mm. in diameter."

The larger part of the following explanation of the construction of sand-clay roads was prepared by W. S. Keller, State Engineer of Alabama, where many miles of sand-clay roads have been built and are giving good satisfaction.

"Every farmer who lives in a section of country where both sand and clay are prevalent is more than likely traveling over a section of natural sand-clay road but is ignorant of the fact. He can call to mind some particular spot on the road he travels though it may not be more than 100′ in length, that is always good and rarely requires the attention of the road hands. Good drainage will be noticed at this place, and if he takes the trouble to investigate he will find that a good mixture of sand and clay form the wearing surface. If this 100′ of road is always good, then the entire road can be made like it, provided man will take advantage of the lesson taught by nature and grade the road so that the drainage will be good and surface the balance of the road with the same material. If it is not possible to find the ready-mixed surfacing material convenient to the road, it may be possible to find the two ingredients in close proximity. In case the road after grading shows an excess of sand, clay should be added, or in case clay predominates, sand should be added to produce good results. There are four general ways in which sand-clay roads may be built:

"1. Ready-mixed sand and clay placed on clay, sand, or ordinary foundation.

"2. Sand and clay placed on soil foundation and mixed.

"3. Clay hauled on a sand foundation and mixed with the sand.

"4. Sand hauled on a clay foundation and mixed with clay. Taking up the various methods in order:

"1. A natural mixture of sand and clay can often be found where the two materials are found separate. The most important point is to know the natural mixture when seen. The very best guide to this is to find a natural piece of good road. A sample from the best of this section will, by comparison, indicate what is required, close to the road to be surfaced. This natural mixture of sand and clay can be noticed where red clay and sand crop out, usually well up in the hills, having ditches and cuts the appearance of red sandstone. A good stratum of well-mixed sand and clay will stand perpendicular in cuts and ditches, resisting erosion almost as well as sandstone. A test of the best natural sand-clay mixtures will show the sand forms about 70% of the whole. The test is very simple. Take an ordinary medicine glass, measure 2 oz. of the mixture into the glass, and wash out the clay. Dry the remaining sand and measure again on the medicine glass. The loss will be the amount of clay originally contained in the mass.

"Before placing any sand clay on the road, the road should be graded to the desired width. The surface of the graded road should be flat or slightly convex. The sand clay should be put on from 8 to 12" in thickness, depending on the character of the subgrade or foundation. With a hard clay for foundation, 8" of sand clay will suffice. If the subgrade is sand, it is well to put on as much as 12" of the surfacing material. After a few hundred feet of surfacing material has been placed, a grading machine should be run over it to smooth and crown the road surface before the top becomes hard and resists the cutting of the blade. It is a good plan to turn the blade of the machine so as to trim the edges of the surface part, discharging the excess sand and clay onto the earth shoulders. After one round-trip with the blade turned out, the remaining dress work with the machine should be with the blade turned in, with the exception of one trip down the center of road with the blade at right angles to the axis of the road for the purposes of distributing any excess of material left in the center.

"After the machine work, it is well to follow with a drag, which smooths any rough places left by the machine and leaves the road with a smooth, even surface. A sand-clay road, unlike other roads, cannot be finished in a short space of time. It can be left in an apparently finished condition with a hard, smooth surface, but it will be found on close examination that the hard surface is, in reality, only a crust, below which there are several inches of loose material. After the first hard rain the crust softens, the road becomes bad, and the work appears to be a failure. This, however, is just what is needed to make it eventually good. After the surface has dried until the mass is in a plastic state, it should be dragged until the surface is once more smooth, with proper crown, and should be kept this way by dragging at least once a day until the sun has baked it hard and firm. The mistake of keeping traffic off during this process of resetting should not be made. The continuous tamping of the wheels of wagons and hoofs of horses is just what is needed to compact the sand clay into a homogeneous mass. The ordinary roller is not very effective in this work, but corrugated rollers have given excellent results. One type which is widely used has 18 cast-iron wheels weighing 300 lb. each, which compress the bottom of the mixture first. As the material becomes more and more compact the wheels ride higher and finally the surface is so hard that the roller does not sink into it it all. A drag is an indispensable machine in the construction of any kind of sand-clay road.

"2. Sand and clay placed on a soil foundation and mixed. This is necessary where the old road had neither a sand nor clay foundation and it is impossible to find the two ingredients ready mixed, but possible to get both in separate state near at hand. The clay should first be placed on the road to a depth of 4" and the required width. It is not wise to place more than a few hundred lineal feet of clay before the sand is hauled, as the clay rapidly hardens and makes the mixing process difficult. After, say, 400' of clay have been placed, the clay should be broken by means of a plow and harrow, if it has become hard, and sand to a depth of 6" placed on it. This should be plowed and harrowed in thoroughly. This is best done immediately after a rain, as the two can be more satisfactorily mixed. The traffic aids the mixing and should be encouraged on the road. After the mass appears to be well mixed, the road should be properly shaped, as previously explained. The road should be given watchful attention and, should sand or mud holes appear, a second plowing and mixing should be given it.

"3. Clay hauled on a sand foundation and mixed with sand. The mixing process is similar to that described under the second head. It is only necessary to add that, as the foundation is sand, a little more clay will be necessary than where the foundation is of clay or soil.

"4. Sand hauled on a clay foundation and mixed with clay. The clay foundation should be plowed to a depth of 4" and harrowed with a disk or tooth harrow until the lumps are thoroughly broken or pulverized. Sand should then be added to a depth of 6" and mixed as before described.

"Sand and clay can be mixed best when wet, but as most road construction is done in the summer months, it is necessary to do most of the mixing dry and keep the road in shape after the first two or three rains, while the passing wagons and vehicles give the road final wet mixing. A sand-clay road is the cheapest road to maintain, since it can be repaired with its own material. with a drag or grading machine, ruts can be filled with material scraped from the edges, whereas on gravel or macadam roads this is not possible. The repairing of these roads can be done almost exclusively with the drag, only enough hand work being required to keep the gutters open and the growth of weeds cut on the shoulders. Holes are repaired by adding more sand clay and when many of them appear, fresh sand clay should be spread over the surface of the road. If the road gets into really bad condition, the roadbed should be plowed up, reshaped, and fresh sand clay added. This is unnecessary where the road is maintained properly and the travel is not too heavy for this type of construction."

Top Soil or Sand-clay Surface (Alabama State Specifications)

"**Description.**—Upon the subgrade, prepared as hereinbefore specified, shall be constructed a top soil or sand-clay surface of the cross-section and compacted thickness shown on the plans.

"**Surfacing Material.**—The surfacing material shall consist of top soil or natural sand clay obtained from the fields or pits designated by the engineer, and as near the right of way as practicable, or, in the event that it is impracticable to secure suitable top soil or natural-mixed sand clay, the surface shall consist of an artificial mixture of sand and clay, the materials for which artificial mixture shall be obtained from places designated by the engineer. Before any surfacing material is used it shall first have been approved by the engineer. The surfacing material shall be free from trash or other foreign matter and contain no stones or boulders that would fail to pass a 1½" ring. Should any such non-road-building material be placed on the road, it shall be removed by the contractor at his own expense.

"The fields or pits from which the surfacing material is to be obtained will be furnished by the county free of charge to the contractor, but the contractor must provide and maintain at his own expense all necessary roads for hauling the surfacing material to the road.

"**Construction Methods.** *Case* 1.—Where the surfacing material consists of either top soil or natural sand clay that has been approved for use without the admixture of any other material, it shall be evenly spread on the subgrade to such depth that, when compacted, the surface will have the compacted thickness shown on the plans. The material shall be dumped on the subgrade in longitudinal rows containing not more than ⅛ cu. yd. to 10 lineal ft. and the number of rows shall be such that when the material is spread the desired cross-section and thickness of surface will be obtained. After sufficient material has been dumped in this way for 100 to 200' of road surface, and before any part of the rows has commenced to pack, it shall be spread approximately to the required cross-section and harrowed to secure uniformity. The spreading may be done by hand or with a road machine in the following manner: (1) The machine shall be run over the road with the blade set so as to scrape off the tops of the piles and fill in the spaces between. (2) The outside edges shall be gone over with the blade set so as to pull the top soil or sand clay toward the center. (3) The grading machine shall be run over the shoulders so as to pull the shoulder material up against the surfacing material and thus bring the entire road surface approximately to the required cross-section. The surface portion of the road shall then be harrowed with either a tooth or disc harrow until uniform density is secured, after which the road shall be brought to the required cross-section and so maintained until accepted. The shaping or reshaping of the surface shall be undertaken only when the weather conditions are such that the loosened surfacing or shoulder material will be readily compacted by traffic to form a well-bonded surface.

"*Case* 2.—When the surfacing material is to consist of an artificial mixture of sand and clay, made by mixing the material of the roadbed with sand or clay from some other source, the construction shall proceed in the following manner:

"1. The surface of the roadbed shall be thoroughly loosened by plowing and harrowing to a depth of from 4 to 8″, according to the nature of the two materials to be mixed, and as the engineer may direct.

"2. The material to be added shall be dumped and spread in the manner described for Case 1.

"3. The added material shall be thoroughly mixed and incorporated with the material of the roadbed. The mixing shall be done by means of plowing and harrowing and shall continue until the engineer is satisfied that the two materials are thoroughly mixed in proper proportion. A part of this mixing shall be done when the road is wet so that the surface will be puddled.

"4. If, after mixing the two materials as described above, a deficiency of the added material is apparent at any point, such deficiency shall be immediately corrected by spreading more of the material at that point and continuing the mixing as above described.

"5. After the mixing is complete, as above specified, the road shall be shaped and maintained as provided in Case 1, except that wherever a poor mixture is observed it sha'l be corrected by additional mixing or by adding necessary material and mixing.

"*Case* 3.—When the surfacing material is to consist of an artificial mixture of sand and clay, both of which materials are to be obtained from without the road, the construction shall proceed in the following manner:

"1. The materials shall be spread in successive layers on the road and mixed in place. The engineer will determine the order in which the two materials shall be spread, as well as the depth of layer for each material.

"2. The dumping and spreading of the materials shall be done as specified for Case 1.

"3. The mixing of the two materials and the maintaining of the road shall be done as specified for Case 2.

"**Basis of Payment.**—The contract price per cubic yard for top soil or natural sand-clay surfacing material shall be full compensation for loosening, loading, spreading, and harrowing, and for hauling the surfacing material 1 mile or less, as well as for shaping and maintaining the surface true to cross-section until the road is accepted. In the case of an artificially mixed surface the materials brought onto the road will be paid for at the contract price per cubic yard for top soil or natural sand-clay surfacing material, which will include all the items mentioned above except harrowing, and the contract price per square yard for mixing will be full compensation for all necessary harrowing, plowing, or other mixing. Measurement of all top soil or sand-clay surfacing materials will be made in trucks, wagons, or cars, as it is delivered on the road, except that where the surfacing material is hauled in scrapers or shoveled on to the road it will be measured compacted in place on the road.

"Stripping of surfacing material pits will be paid for at the contract price for common excavation.

"The bid price for overhaul of sand clay will be allowed on all sand-clay materials necessarily hauled a greater distance than 1 mile. The method of determining overhaul will be in all respects similar to that provided for determining overhaul in the case of crushed-stone surfaces."

GRAVEL ROADS

A coarse, well-graded gravel is the most satisfactory material for a cheap year-round road. It gives body to the traveled track, binds well, rides easily, and with a consolidated depth of 8 to 20″ holds all ordinary loads after it is well consolidated. For wheel pressures and depths of metaling see Chap. VI (p. 391).

Cost.—Pit-run gravel varies in cost from 75 cts. to $3.50 per consolidated cubic yard in place, screened gravel from $1.50 to $4 per consolidated cubic yard. Gravel surfacing adds approximately $2000 to $6000 per mile to the cost of an earth road in the same location and a fair comparative price for this type, including drainage and incidentals, ranges from \$4000 to \$10,000 per mile (1926 cost conditions). Maintenance is discussed in Chap. VII. The cost has a wide range from \$100 to \$600, depending on the volume of traffic and degree of perfection of maintenance. As a

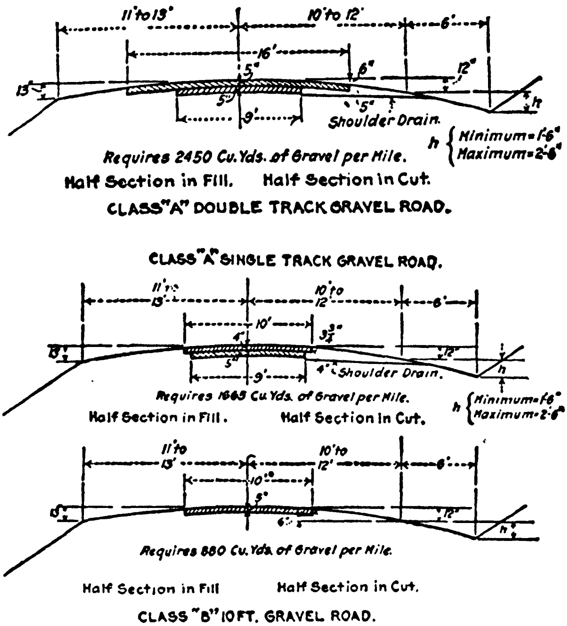


FIG. 115.—Gravel roads State of Iowa.

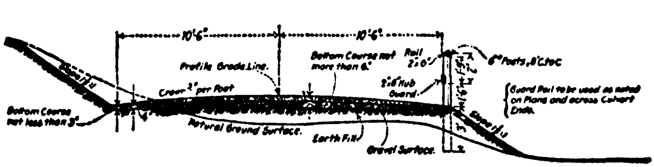


FIG. 116.—New Hampshire gravel road section.

general rule, where the maintenance cost exceeds \$400 to \$600 per year it is advisable to build a better grade of pavement.

General Suitability.—At the present time 50% of the mileage of surfaced roads in the United States are gravel roads. They are suitable for volumes of traffic up to 350 to 400 vehicles daily.

Beyond this limit they often become rough on account of so-called rhythmic corrugations which are difficult to control. Gravel has been utilized as a temporary expedient up to 2000 to 3000 vehicles daily, but it cannot be advised for any such amount of traffic. Oiling with a light cold asphaltic oil or cold tar is resorted to under

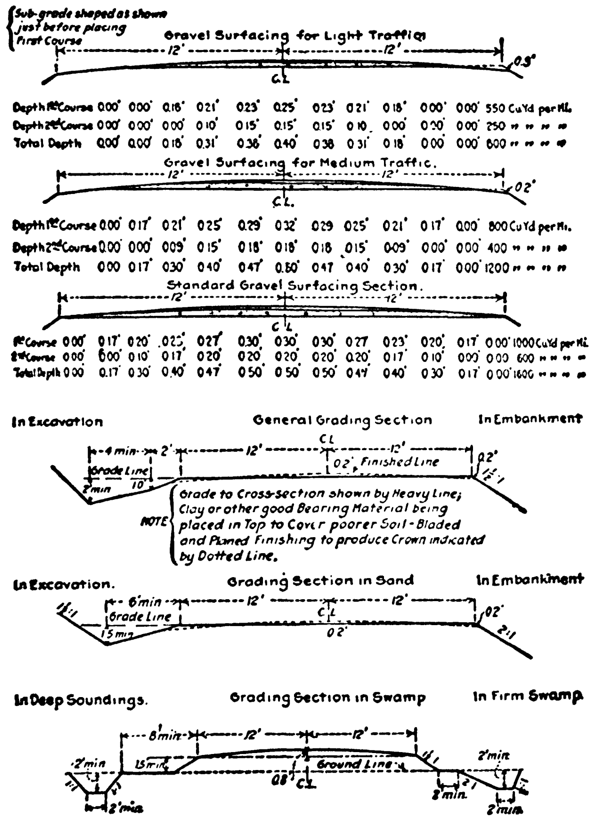


FIG. 117.—Typical gravel road section, State of Minnesota.

moderately heavy automobile traffic. No gravel road should be oiled till at least a year old, so that it may be completely consolidated and firmly bound. The surface must be well cleaned of excess fine dust and the oil applied in two or three successive light coats of approximately ⅛ gal. per square yard at intervals of 2 or 3 months. It takes more than one application to give even moderately good results, as the clay and loam in the road tend to prevent the formation of a good bond between the oil and gravel, but if persistent treatment is adopted this method increases the power of gravel roads to withstand touring-car traffic, but, of course, does not increase their structural strength or make them suitable for heavy-unit freight hauling. An official report on page 360 illustrates the economic use of gravel as compared with higher-priced pavements for special conditions even on primary state highways. Figures 115 to 117 and Chap. III (p. 147) show usual practice. Table 74 (p. 391) gives required depths for different loads on different soils.

Construction Essentials.—Gravel roads are hard to consolidate quickly and need careful and continuous attention to prevent the formation of ruts, holes, or humps. They cannot be built by merely dumping loose gravel on the road and then hoping that traffic will put it in shape. A large mileage has been built on this principle and the results are shameful. A successful gravel road requires careful selection of the gravel, careful spreading, careful consolidation, and constant maintenance. Current practice is shown in typical Specifications (pp. 357 and 1452); the essential features will be summarized at this point.

Size of Gravel

Gravels suitable for road work are widely distributed over the country. They occur in bank deposits and in stream beds. The prime requisite of a gravel for foundation courses is that it contain a large percentage of coarse pebbles to give body and distribute the wheel loads. The prime requisite for a surfacing gravel is hardness of the stone and well-graded coarse and fine particles which will take the wear evenly and bond well. Pit-run gravel varies greatly as to size and composition even in a single pit, and for this reason no definite limits can be well set for the proportion of sizing. In general, it can be said that for foundation courses any coarse gravel, which when screened through a ¼″ mesh contains less material passing the screen than retained on it, can be successfully manipulated without screening to remove the excess sand. In some localities this limit is not feasible on account of excessive fine material and the limit of fine material passing a ⅛″ mesh is placed at 60%, but, in reality, a gravel of this fineness does not produce satisfactory results and a road on which it is used becomes more nearly a sand-clay construction than a gravel type. For a top course the large stone above 1½″ in size should be screened out, and if pit-run gravel is used the sand passing the ¼″ mesh should not exceed 40% of the volume. The most satisfactory top is a screened gravel, but this adds materially to the cost. Where screened gravel is used ½ to 3″ is satisfactory size for the bottom course and ½ to 1½″ for the top course with a light surface wearing coat of sand ranging from pea size up to ½″ in diameter.[1]

[1] There is some tendency to reduce maximum size of top-course gravel to 1¼ or even 1″.

The following specification has been recommended by the Committee on Materials of the A.S.C.E.

"Two mixtures of gravel, sand, and clay shall be used, hereinafter designated in these specifications as No. 1 product (for top course) and No. 2 product (for middle and bottom courses).

"Number 1 product shall consist of a mixture of gravel, sand, and clay, with the proportions of the various sizes as follows: all to pass a 1½" screen and to have at least 60 and not more than 75 % retained on a ¼" screen; at least 25 and not more than 75 % of the total coarse aggregate (material over ¼" in size) to be retained on a ¾" screen; at least 65 and not more than 85 % of the total fine aggregate (material under ¼" in size) to be retained on a 200-mesh sieve.

"Number 2 product shall consist of a mixture of gravel, sand, and clay, with the proportions of the various sizes as follows: all to pass a 2½" screen and to have at least 60 and not more than 75 % of the total coarse aggregate to be retained on a 1" screen, at least 65 and not more than 85 % of the total fine aggregate to be retained on a 200-mesh sieve."

Bonding Properties.—Clean gravel will not bond well. A small percentage of clay, loam, or lime dust is desirable and necessary. This per cent ranges from 10 to 20%. For bottom course, pit-run gravel which contains over 20% of clay or loam should not be used; from 10 to 15% gives the best results. For top course, 10% is about the maximum clay or loam allowable, particularly if it is to be treated with bitumen. Many so-called cementitious gravels of lime rock contain or produce under traffic a first-class rock-dust binder of the highest grade. Clay or loam can be added to a clean gravel by mixing at the pit or by placing a thin layer of such material over the gravel as spread on the road and mixing it with the course during consolidation.

Spreading.—Gravel must be uniformly spread; there are two general methods—the trench spread (Fig. 115) and the feather-edge spread (Figs. 116 and 117). The trench spread is not so likely to break down at the edges as the feather-edge, but this can be remedied by a wider spread where the feather-edge method is used. In either case the depth should be uniform and the surface properly crowned. Gravel should not be dumped in piles; it should be spread along in windrows or in sheets with Manger spreader and the spreading finished by shoveling, raking, or by road machine blade scrapers. If pit-run gravel is used, the course should be harrowed to distribute the sizes uniformly. The ratio of compacted to loose depth is approximately 1.2 or 1.25. that is, a loose depth of 8" will compact to about 6½". If screened gravel is used, the filler should be added before the course is rolled.

Consolidation.—Consolidation is the hardest feature of pit-run gravel construction. Detail methods are described under Gravel Foundations (p. 432). A combination of traffic and roller consolidation while the gravel is moist gives the best and quickest consolidation, although traffic alone will put it down firmly if given time and the shape is kept intact by constant dragging with a hone or road machine. The Minnesota Specifications show the method employed where a road roller is not used.

Minnesota Specifications

Graveling

"**Description.**—Graveling shall be construed to mean all surfacing with pit-run gravel, screened gravel, or crushed rock, or crushed rock screenings built in two or more successive courses.

"**Materials.**—All materials shall be of a quality approved by the engineer and shall be the best obtainable from the specified pit or quarry. Materials for the first course shall contain no stone which would be retained on a screen having 2½" openings. Materials for the second course shall contain no stone which would be retained on a screen having 1" openings. If available material contains an excess of sand, such excess shall be handled as provided by special specification for each job or project.

"**Subgrade.**—The cross-section of the subgrade shall be shown on the standard cross-section accompanying the plans. Graveling upon a wet muddy roadbed will not be permitted. If the graveling is not done in conjunction with the grading as a part of the same contract, the subgrade for the full length of the job embraced in the graveling contract shall, before being graveled, be dressed by the county to the cross-section above mentioned. Thereafter, the contractor shall keep it dressed to the specified cross-section and free from ruts, waves, and undulations, as part of the graveling contract. If the grading and graveling are performed under the same contract, the preparation of the subgrade shall be performed as part of the grading item and no additional charge will be allowed therefor under the graveling.

"**Loading and Hauling.**—Loading from pits shall be performed in such a manner and by such methods that a uniform grade of materials will not be delivered upon the road. Stone exceeding the sizes specified shall not be loaded. No earth, sod, or any foreign or vegetable matter, nor an excess of sand or clay, will be allowed in the gravel, and care must be taken that strippings be not mixed with the gravel. Any loads taken to the work containing such objectionable materials will be rejected.

"**Dumping and Spreading First Course.**—The first-course material shall be deposited in a uniform ridge on the center line of the road and shall be spread immediately upon the subgrade to a uniform section. This work shall be started at a point on the road nearest the pit or loading place and shall proceed therefrom until the extreme haul in that direction is reached.

"**Shaping and Compacting.**—The resurfacing material shall be shaped, while being compacted under travel by the use of a blade grader, tooth harrow, planer, or other suitable means. Ruts formed by the hauling or by travel shall be dragged full at least once each day, and more frequently if necessary, to prevent cutting through the surfacing material into the subgrade. Holes, waves, and undulations which develop and are not filled by dragging shall be filled by adding more material according to the direction of the engineer. The shaping of the material shall be performed according to the direction of the engineer and shall be continued until the material is well compacted, free from ruts, waves, and undulations, and is made to conform to the cross-section indicated on the standard above mentioned.

"If the material is not sufficiently compacted by the above methods within 20 days after placing, the engineer shall direct the character, amount, and method of applying the binding material necessary to provide a compacted surface, and the contractor shall provide the necessary labor and equipment to perform such additional work at the unit prices submitted for application of the regular surfacing material. The county shall furnish the binding material in the same manner as provided for the regular first- and second-course material.

"**Second Course.**—When the first course is compacted and shaped as specified, to the satisfaction of the engineer, he shall authorize the application of the second-course materials. It shall then be applied, shaped, and compacted by the methods specified for the first course. The work of shaping and compacting shall be continued until the material is well compacted with the surfacing free from ruts, waves, and undulations and conforming to the specified cross-section."

Other Coarse Materials.—The same principles apply to the use of any available local material, such as slag, chert, caliche, disintegrated granite, shell, etc., each one of which can be used to advantage in special localities. A specification for chert follows:

CHERT SURFACE (ALABAMA STATE SPECIFICATIONS)

"**Description.**—On the subgrade, prepared as hereinbefore specified, shall be constructed a chert surface of the cross-section and compacted thickness shown on the plans.

"**Chert.**—Before any chert is used in the construction of the road surface, it shall first have been approved by the engineer. In general, the special provisions accompanying the proposal form will indicate the locations from which satisfactory chert may be obtained and also indicate whether crushing and screening of the chert will be required. Any large particle of chert that may be spread on the road, and that would fail to pass a 2" ring, shall be broken by hammering, or otherwise, before the surface is rolled. Unless otherwise provided in the special provisions, chert quarries will be furnished by the county free of charge to the contractor, but the contractor must provide and maintain roads to such quarries at his own expense.

"**Construction Methods.**—The chert shall be spread on the subgrade to such depth that, when compacted, the surface will have the compacted thickness shown on the plans. It shall be dumped, spread, and rolled in the manner hereinbefore specified for dumping, spreading, and rolling No. 1 and No. 2 crushed stone in the case of crushed-stone surfaces. Where, in the judgment of the engineer, the conditions are such that a well-bonded surface is not secured by rolling as specified above, the contractor shall sprinkle the surface with water while rolling is in progress, and the sprinkling shall be done in such quantity and in such manner as the engineer may direct.

"**Basis of Pavement.**—The contract price per cubic yard for chert for surfacing shall be full compensation for quarrying, crushing, loading, spreading, rolling, sprinkling, hauling the chert 1 mile or less, and maintaining the surface until the road is accepted. Measurement of chert will be made in trucks, wagons, or cars as the chert is delivered. Stripping of chert quarries will be paid for at the contract price for common excavation. The bid price for overhaul of chert will be paid on all chert necessarily hauled a distance greater than 1 mile. The method of determining overhaul will be in all respects similar to that provided for determining overhaul in the case of crushed-stone surfaces."

REPORT ON RHYTHMIC CORRUGATIONS

By George E. Ladd, for U. S. Office of Roads

"**Their Relation to Traffic.**—Gravel roads subject to a traffic of not more than 200 or 300 cars per day remain practically free from corrugations, if occasionally dragged. As soon, however, as traffic reaches 400 to 450 cars per day, corrugations develop very rapidly. In the state of Maine, where tourist traffic is large during summer months, and its rate of increase is well known, the state highway officials can predict almost to a day when corrugations will begin to develop on certain roads. The traffic limitations of gravel roads, in general, are determined by the intensity of maintenance required and by its costs. Sufficient data have not yet been gathered on this subject. A highway engineer's handbook covering the subject of gravel roads states that they are not fit for traffic exceeding 100 cars per day, but in Maine, Connecticut, New Hampshire, and other states they are successfully serving a traffic of from 700 to several thousand cars per day. In Wisconsin, gravel roads are maintained with comparatively smooth surface so that auto travel is comfortable and satisfactory on roads serving from 1800 to 3200 cars per day with a maintenance cost ranging from $280 to $320 per year, or from $600 to $700 per year when the roads are also treated with dust preventives.

"Maintenance costs on gravel roads in various states have been reported to the writer as ranging from $1000 to $3000 per year. No satisfactory analyses of such costs have been made and it is presumed that a considerable portion of them really belong to improvement and reconstruction.

The Relation of Rhythmic Corrugations to Kinds of Gravel

"**Mix and Methods of Construction.**—The fact that these corrugations are so general and occur in all sections of the United States makes it evident that nearly all gravel roads are subject to the development of this nuisance and menace, if they serve sufficient traffic, although the methods of construction and kinds of gravel vary widely. The situation, however, is not hopeless. Several highway officials have expressed the opinion that less corrugation trouble is found where the gravel is angular and where it is composed of highly rounded particles. It is also claimed that where the road is constructed of gravel more uniform in size than pit run, and especially where everything over 1" is excluded, that corrugations are slower in developing and more easily eliminated by maintenance methods. In northern New Hampshire, some roads are built of so-called gravel which has resulted from the decomposition of granite rocks and is full of angular quartz. The roads built of this material are said to give better service so far as corrugations are concerned than those, built in the same state, of glacial gravels, or those resulting from disintegration of schistose rock. It is claimed in Wisconsin that the best service given by gravel roads, so far as corrugations are concerned, is found where pit-run gravel is passed through a crusher and only the material which ranges in size from 1" down is used. Highway engineers, notably some in Wisconsin and in Oregon, object to any clay binder in the gravel and prefer only the fines produced by crushing and such as result from surface wear.

"During the writer's investigation in the state of Maine it was noted that roads with a relatively high per cent of clay-silt binder were comparatively free from corrugations. On one road on which rhythmic corrugations were well developed a patch near the center of the road, about 70' long and varying from 1 to 4' in width, was uncrossed by corrugations, and had a hard, smooth surface. Examination showed that this patch had about 5 % more clay-silt bond than the surrounding road.

"On the gravel road between Waterville and Bangor corrugations develop rapidly during the summer traffic, and are kept down only by constant maintenance for almost its entire length. Comfortable travel on this road was made possible only by continuous use of drags and planers. One short section of the road, however, was practically free from corrugations throughout the summer. This section was built of a softer gravel, which was high in clay-silt bond. Unfortunately, though, while this section of the road gives satisfaction during the summer months, it is said to be nearly impassable in the spring. Thus it may be that an all-year gravel road that will not require intensive maintenance cannot be built in states subject to heavy freezing and wet seasons.

"This problem remains for future solution and must be worked out by a combination of field observations and laboratory tests. In this connection it may be stated that the present standard tests of gravel are unsatisfactory in that they do not include determination of the nature and qualities of natural binder originally in the gravel. The so-called cementing-value test has to do only with the products of abrasion of individual gravel particles. While it is important to determine this factor, there must also be a determination, in addition to the facts brought out by mechanical analyses as to the quantity of clay-silt binder, of the cementing value of this fine material. The writer is now engaged in experiments connected with the testing of different mixtures of gravel, sand and clay-silt binders, and their behavior under different degrees of compacting and with different amounts of water present. In future investigations, observations must be made over a wide area and under varying conditions as to the relation between the amount and kind of natural binder in the road and the behavior of the road under traffic. Another interesting problem in this connection is the question of migration of clay binders toward the surface, and their ultimate loss to the road through the combination of rain, wind, and traffic influences.

"**The Effect of Dust Preventives and Surface Binders.**—A preliminary study had been made of problems of surface-treated gravel roads. Matters of methods, costs, and service have been studied to some extent in certain states. Surface binders, in a general way, prevent development of corrugations, and where these do not occur on binder-treated roads, it has been demonstrated that they result from excessive quantities of bituminous material. This statement does not refer to dust-prevention treatment, and is perhaps not true of glutrin-treated roads. Some states use both tar and oil binders, some will use only oil, and others only tar. One state, at least, uses glutrin almost exclusively. Dust preventives have been seen to fulfil the primary requirement. Their effect on corrugations, however, is not yet clear. It is not unlikely that they delay the development of rhythmic corrugations of the first type discussed above. It is clear, however, that they do not prevent their development. They successfully lay dust for long periods, in spite of heavy traffic, and they lengthen the life of a road by retarding loss of surface material.

"**Maintenance Methods.**[1]—The maintenance required by a gravel road depends, of course, largely upon the amount of traffic to which it is subjected. If this does not exceed 200 to 300 cars per day, occasional dragging, especially after rains, will keep the surface in good condition. As traffic increases, however, surface maintenance must be practically continuous, and even then dragging or scraping will not keep a road free from corrugations. The planer, a simple machine familiar to all highway engineers, is employed in addition to the drag and road scraper. Planers[2] are of different types. Those used in some states have a single cutting blade, while those used in other states have several shorter blades so arranged as not to cut down corrugations but to distribute and rearrange the surface materials. Apparently, the best machine for eliminating corrugations has not been devised. The planer certainly does not always remove them, some of the bases of ridges being left so that traffic may, in 24 hr. after the planing of the road, reproduce the corrugations in original form and position. Even resurfacing with 1½ to 2" of gravel, following planing of the road, will not prevent the immediate recurrence of corrugations which have not been entirely eliminated by planing and dragging. In Wisconsin, some gravel roads subject to heavy traffic are scarified once or twice during the summer months, and left to be recompacted by traffic. This is heroic treatment. Experiments are contemplated for this coming field season which may lead to improving maintenance methods, so far as corrugations are concerned. The whole subject of maintenance of gravel roads, and its costs and economic significance, will be discussed in another paper."

The following condensed report shows a case where gravel construction was recommended on a primary state road.

FEDERAL AID PROJECT 184 WOODVILLE—BRISTOL SPRINGS—CHESHIRE

"**Alternate Estimates.**—Three alternate estimates are submitted. Any of these will serve the purpose and each has certain desirable features.

"**First Estimate, Reinforced Cement Concrete 7", 6", and 7" Mesh and Bar.**—This design is estimated at $620,000, or an increase of approximately $154,000 over the estimate of August, 1922. Approximately $36,000 of this increase is due to relocation changes requested by the Office of Public Roads. Approximately $118,000 of the increase is due to change in pavement thickness, guard-rail type, culvert improvements, and increase in unit prices.

"This design will satisfy the Office of Public Roads but will not satisfy the local authorities, due to large increase in cost.

"As per our previous correspondence, there is some doubt as to the advisability of attempting to construct a concrete pavement on the fresh high fills and new side-hill grading, Sta. 0–240, for a series of short breaks aggregating from 3000 to 7000 lineal feet as per list attached to alternate typical section.

"**Second Estimate, Oiled-gravel Pavement.**—This design is estimated at $450,000 as compared with the August estimate of $466,000. This design is based on a 13" oil-gravel pavement with extra subbase for the entire length of the road. It also used guide posts as per original estimate on straight alignment.

"This design will satisfy the Office of Public Roads and the local authorities. It will increase the yearly maintenance charge for the first few years until recapped, but, considering the volume of traffic, this will not be beyond the bounds of reason, as indicated by our maintenance records for this type under light traffic. This design seems to be the most rational solution for this particular road, considering all the factors involved, namely, high cost of grading, impossible to eliminate and still get the approval of the U. S.

[1] Mr. Ladd's report bears out local experience that maintenance costs increase very rapidly where the volume of traffic exceeds 450 to 500 daily average.

[2] See p. 571, Chap. VII.

Bureau; second, limited local funds; third, economy of final cost as compared with concrete-pavement construction.

"**Third Estimate, Combination of Concrete and Oiled Gravel.**—This design is estimated at $584,000, or an increase in cost of $118,000 over the estimate of August, 1922. This design substitutes a 13" oiled-gravel pavement for cement concrete on the doubtful grading sections of the road. It also used guide posts in place of cable rail on straight alignment.

"This design will satisfy the Office of Public Roads and may serve as a compromise solution with the local authorities, provided the Department desires to use concrete on this road.

"**Pavement Type.**—This road is isolated from railroad shipping points and imported materials would be prohibitive in cost. There is a limited amount of local boulder stone, 7000 to 10,000 cu. yd., widely scattered and difficult to collect, Sta. 240–519. This stone is suitable for boulder base. There is one good gravel pit of unlimited capacity on road (Long's pit, 1600′ from Sta. 140). This gravel is suitable for gravel foundations and screened gravel top course and, if washed, for cement concrete pavement. This gravel seems to be the most feasible source of supply. Alternate estimates have been prepared for cement concrete pavement 7", 6", 7" costing approximately $3.40 per square yard and for a 13" oiled-gravel pavement costing approximately $1.50 per square yard. There are 86,000 sq. yd. of pavement on this job.

Cement Concrete 7", 6", 7" Mesh and Bar

Cost per square yard

0.18518 cu. yd. concrete at $7.90	$1.46
0.3518 bbl. cement at $3.90	1.37
9 sq. ft. mesh at 5.5 cts	0.50
2.81 lb. bars at 3.6 cts	0.10
	$3.43

86,000 × $3.40 = $292,400.
Average maintenance, 0.8 cts. per square yard.
Class III traffic (less than 500 vehicles daily).

Gravel Pavement Oiled 13" Depth 6", 4", 3"

Cost per square yard

0.278 cu. yd. pit-run foundation at $3.40	$0.95
0.083 cu. yd. screened top at $5.20	0.43
0.37 gal. oil at 15 cts	0.06
3 lb. calcium chloride at 1.2 cts	0.04
	$1.48

86,000 × $1.50 = $129,000.
Average maintenance, 6.5 cts.
Class III traffic (less than 500 vehicles daily). Under these conditions the gravel pavement seems to be a desirable solution.

W. G. Harger,
Designing Engineer.

MISCELLANEOUS SPECIAL CASES

Alaskan Trails and Roads.—Alaskan climatic and soil requirements afford special problems; the following quotation from *Engineering and Contracting* of Mar. 6, 1918, indicates an interesting condition as described in the report of the Alaskan Highway Commission:

"The most unusual and troublesome feature encountered in construction is the permanently frozen ground which covers a large portion of the entire interior, and which is protected from thawing during the summer by a thick layer of moss, turf, or decayed vegetable matter. The character of this frozen material varies largely in different sections of the territory, and even in the same section. It may be gravel, clay, silt, peat, or clear ice, or a combination of two or more of these elements.

"When gravel is encountered the problem presents no special difficulties; the moss or turf is stripped off, and the road graded in the usual manner. When the material is clay, experience has shown that the same procedure can usually be followed, but the grading is a slow and rather expensive process. After the protective covering of vegetable matter is removed, it is necessary to allow the soil to thaw and dry out somewhat before it can be worked, and unless a considerable period is allowed to elapse between the stripping and the grading, it will be found that the thawing is not extended to sufficient depth to permit of completing the grading in one operation. When the necessity for the road is not pressing, an appreciable saving can be effected by stripping the roadbed and digging drainage ditches during one season, completing the construction the next year.

"In those localities, however, where the frozen material is silt or peat the stripping of the roadbed quickly results in the formation of a quagmire through which a man or horse, even without a load, can pass only with the greatest difficulty. Such soil has sufficient bearing value only as long as it remains frozen, which makes it desirable that the moss or turf overlying it be kept intact. This layer of vegetable matter is not of itself able to sustain traffic, necessitating the addition of a protective covering—usually of scrub spruce timber which covers a large part of interior Alaska, except the Seward Peninsula, and affords excellent material for this corduroy.

"Where the trees are large enough, pole corduroy is constructed by grubbing all stumps and roots from the roadbed, leveling it and laying perpendicularly to the axis of the road a single layer of poles from which the largest and stiffest branches have been trimmed. Ditches are then dug at a distance of 3 to 5′ from the ends of the poles, and the material therefrom, after rejecting the top layer of vegetable matter, is placed on the corduroy for the double purpose of protecting it from wear and affording a smoother roadway. If the soil in the ditches is entirely unsuitable for this covering, other material, preferably gravel, is hauled on from the nearest available source.

"Where the spruce timber is of very small size, or where only small willows are available, as on the Seward Peninsula, brush corduroy is used. The method of construction is similar to that described above, except that the single layer of poles is replaced by a mattress of untrimmed brush containing sufficient material to give a thickness of at least 6″ when compressed.

"When corduroy has been properly protected, its life in most parts of Alaska is quite long. Poles taken out of the road after 10 years of service have been found to be in excellent condition.

"The 3′ to 5′ berm which is left between the ends of the corduroy and the ditches is very necessary to protect the corduroy from undermining, as the ditches, under the action of sun and rain, slough and cut rapidly. Ordinarily, as the frozen soil thaws and cuts away the moss, the berm gradually assumes a gentle slope to the bottom of the ditch, effectually protecting the corduroy, but where the cutting is severe, it often becomes necessary to revet the insides of the ditches with moss or turf. Frequent outlets from the ditches must be provided, and when the amount of water reaching the ditch on the upper side of the road is large it is advisable to construct an additional ditch parallel to the road and about 50′ away, with sufficient outlets to culverts of ample size.

"Along the Pacific coast of Alaska no frozen ground is encountered, but the mountainous character of the country, the excessive rainfall, and the difficulties of clearing have made the work, as a rule, even more expensive than in the interior. Unless the soil encountered in this region is gravel, it will not stand up under traffic during the heavy and continuous rains, and some protective covering is required. Fortunately, gravel is usually found at no great distance; otherwise plank or corduroy roads are constructed.

"The numerous swift streams of glacial origin found in the Pacific coast section and throughout the Alaskan range in the interior have been the source of much trouble and expense. Flowing through gravel beds varying in width with the volume of water carried up to 2 miles or more, they rarely have any fixed channels. It is by no means uncommon for one of these streams to abandon an old channel and establish itself a new one ½ mile away almost overnight. When warm weather causes rapid melting of snow and ice in the glaciers, these streams become raging torrents of enormous destructive force, and roads paralleling them are in constant danger of being washed away. Numerous methods of bank protection to prevent damage from this cause have been tried, of which the following has proved to be the cheapest and most effective. A layer of loose brush of sufficient length to give the requisite protection is placed on the threatened bank, perpendicular to the current and weighted below the center with stone enveloped in galvanized-wire netting, the whole being anchored in place by wires extending to 'dead-men.' For emergency work when the water is too high to permit of placing the wire netting and rock, the brush is made into fascines enclosing sacks of earth, which are then placed against the threatened bank and wired to it and to each other. This form of protection is easily and quickly constructed and has repeatedly demonstrated its effectiveness.

"As now constructed, the width of wagon roads varies with the formation of the ground and the amount of traffic expected, but, as a general rule, roads graded by other means than the road grader are given a minimum width of 20′ between ditches, and those on which the grader is used a minimum width of 24′. On steep side hills and where rock work is involved, the width is reduced to 10 or 12′. The standard width of clearing is 30′, but this is increased to 60′ where necessary in order to secure the beneficial action of wind and sun on the roadbed.

"Sled roads for winter traffic are cleared for a width of 16′, with all stumps, hummocks, and similar obstacles removed for a width of 8′. They are constructed where the amount of traffic is not large enough to justify a wagon road, where the cost of building a wagon road would be prohibitive, or where the communities along the route are amply served by water transportation during the open season, as is the case with the Fairbanks-Fort Gibson sled road. If it seems probable that future development may demand or justify a wagon road, the location is made for wagon road, in order that work done on the sled road may be of use when the improvement is made.

"Trails designated for travel by dog team in winter or by pack train in summer are given a width of 8′ with all stumps and underbrush cut off as close to the ground as possible.

"In the past, the work of constructing and gradually improving the roads has been so generally intermingled with maintenance operations that a systematic plan for maintenance has not been put

FIG. 118.—Roadbed below natural ground elevation.

into effect, nor would such a plan have been feasible in view of the uncompleted state of the roads. At the present time, however, the condition of parts of the more important roads, notably the Valdez-Fairbanks road, is such as to make practicable their maintenance by dragging. As Alaska has only a very small agricultural population, the method adopted in many states of contracting with farmers adjacent to the road for the necessary dragging cannot be used, but

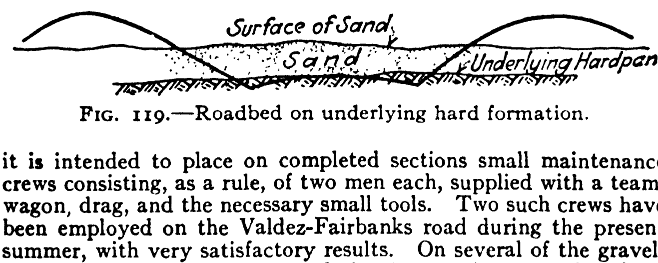


FIG. 119.—Roadbed on underlying hard formation.

it is intended to place on completed sections small maintenance crews consisting, as a rule, of two men each, supplied with a team, wagon, drag, and the necessary small tools. Two such crews have been employed on the Valdez-Fairbanks road during the present summer, with very satisfactory results. On several of the gravel-surfaced roads in southeastern Alaska the patrol system of maintenance has been used in connection with more extensive repairs. The results show the method to be very effective for roads of this character.

"The average costs per mile, including construction and maintenance of all roads and trails constructed by the board since its organization in 1905, are as follows: wagon road, $3419; sled road, $379; trail, $113. A division of these amounts to show the exact cost of construction proper is impossible, but a careful analysis of the available data indicates that the following unit costs of construction, including bridges may be accepted as approximately correct: wagon roads, $2475 per mile; sled roads, $300 per mile; trails, $65 per mile. The average costs of maintenance during the past season were as follows: wagon roads, $250 per mile; sled roads, $41 per mile; trails, $8 per mile."

Arid Regions.—In the arid regions, fills must be avoided. Ordinary earth roads are constructed below the general elevation of the ground which keeps them moist longer (see Figs. 118 and 119); shallow ditches are used for the same reason. In many cases a hardpan formation underlies the sand surface and in these conditions the sand surface is scraped off and the road built on the underlying strata.

Where fills must be used they should be made during the rainy season and the addition of clay to a sandy soil helps consolidate the traveled way. Readers are referred to the reports of the state engineers of New Mexico and Arizona for further data on the special treatment of roads under these conditions.

CHAPTER VI

MACADAM AND RIGID PAVEMENTS

Introduction.—The types of pavement discussed in this chapter cover the forms of construction generally used for main state-aid or country-aid highways ranging in traffic volume from 200 to 20,000 daily. Chapter I outlined economic selection of type, Chapter III, data on widths, crowns, etc. The design of strength and the economic selection of various types of foundation courses are described in this chapter as a preliminary to the discussion of pavement details.

DESIGN OF PAVEMENT STRENGTH

Fundamentals of Pavement Design.—Before discussing in detail the various factors in design, it is, perhaps, just as well to outline the fundamental differences between flexible-pavement and rigid-pavement design; it will be noted that each type has certain well-defined advantages.

Utilization of Subgrade Support.—Macadam or flexible-base pavements move in conjunction with and stay in contact with the underlying earth subgrade. If unequal settlement of the subgrade occurs, the surface becomes uneven under traffic action but continues to transmit the wheel loads directly to the underlying subgrade. These surface inequalities can be easily and cheaply repaired without destroying the effectiveness of the underlying pavement. If large areas settle, the pavement follows the movement without any particular loss of effectiveness. *Macadam design is based on the principle of utilizing subgrade support at all points. The pavement thickness is varied to produce the maximum allowable pressure on the subgrade and in this way utilize the full supporting value of the subgrade.*

Rigid pavements do not conform to the settlement of the subgrade. They bridge over small areas where the underlying soil has settled and they fail entirely where large areas settle unevenly. *Rigid-pavement design is predicated on the lack of soil support for small areas. The pavements are designed to bridge over small areas by means of their slab or beam action (as this is the main advantage of the type), but the cost of applying this method of design for large areas of settlement is prohibitive. Rigid-pavement design cannot utilize soil support to its full value;* that is, a rigid pavement strong enough to bridge over small areas of no support has a needlessly high distributing power (even on poor soils) where the pavement is in actual contact with the subgrade.

The frequency or size of the small areas of no support seem to have very little relation to the relative normal supporting power of different subgrade soils within the usual range of foundation soils. These small areas of no support are due more to non-uniformity of the subgrade, to warping of the slabs due to temperature, and to construction imperfections rather than to low general supporting power of the soil. Some increase in rigid-pavement strength is desirable on clay as compared with sand support, but it certainly is not inversely proportional to the supporting power of the subgrade soils.

The practical effect of this fundamental difference is shown by the wide range in depth required for macadam roads on different soils as compared with the small required range in strength of rigid pavements laid on these same varying soils.

From the standpoint of comparative construction costs, macadam generally has a distinct advantage over rigid pavements where the subgrade supporting power is high, as relatively thin macadam construction is then permitted. It loses this advantage as the soils become poorer in supporting power, necessitating a large increase in macadam depth as against a small increase in rigid-pavement strength. When a really unstable soil where settlement occurs over large areas is reached, however, a rigid pavement is not feasible without the use of underlying macadam base and the macadam type regains its advantage.

TABLE 63.—TABULATION SHOWING APPROXIMATE EFFECT OF FOUNDATION SOILS ON THE RELATIVE CONSTRUCTION COSTS OF TYPICAL BITUMINOUS-MACADAM AND CEMENT-CONCRETE PAVEMENTS UNDER CLASS II TRAFFIC

Foundation soil	Approximate construction cost per square yard (1922)	
	Bituminous macadam	Cement concrete
Coarse sand and gravel	$2.00	$2.80
Loams	2.30	2.90
Clays (ordinary)	$2.60–$3.40	$3.00–$3.20
Wet clay and quicksand	3.00– 3.80	3.60– 4.00

Effect of Temperature and Frost.—*Changes in temperature, except freezing, have no effect on macadam-pavement design.*

Changes in temperature must be considered in rigid-pavement design, as they produce internal stresses, resulting in contraction cracks and "blow-ups," and produce warping, which results in lack of uniform soil support for the slabs.

Frost heave very materially reduces the distributing power of macadam pavements for a short time in the spring by breaking the tight internal lock, and it is at this time that most macadam failures occur, as the subgrade is at its worst at the same time that the pavement has the least distributing power; *that is, in order to prevent* *failures entirely for a short time in the spring, macadam must be designed needlessly thick, considering its distributing power for the balance of the year.* It is prohibitive in cost to design for this extreme combination of unfavorable conditions under heavy traffic on poor soils, and in practice reasonably safe depth is provided, with the idea of minor repair and some restriction on heavy loads for a short time in the spring.

Temperature changes do not affect the internal resisting moment of rigid pavements, which is constant the year round, but temperature changes produce slab distortion, cracks, etc., which increase the normal stress in the pavement. The effect of temperature must always be considered in rigid-pavement design.

Effect of Traffic Loads.—Occasional extremely heavy loads far above the normal vehicle load are not disastrous to the macadam type, as this type of road is more or less self-healing and knits together again under rolling or well-distributed normal load traffic; that is, *macadam roads can be safely designed for the normal maximum vehicle load.*

If a rigid-pavement slab is cracked by exceptionally heavy loads, this crack is not self-healing and it is very difficult to replace a portion of a slab and get reasonable bond with the adjacent pavement; that is, *rigid-pavement design must consider the exceptional load.*

This general fact tends to warrant a difference in design loading for the two types of pavement.

Effect of the Critical Location of Loads.—Critical loading will be discussed in detail later. At this point the discussion will be anticipated by stating general conclusions in order to complete the

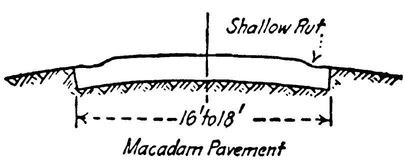


FIG. 120.—Typical shallow ruts along edge of Class I traffic macadam pavement.

comparison of the differences in design fundamentals. On macadam pavements the wheel loads which cause the worst distribution of pressure to the subgrade are located along the edge of the macadam. At this point part of the distributing value of the macadam is lost. This fact does not have much effect where the shoulder is a well-consolidated gravelly soil, but for shifting sand or wet clays it helps cause the formation of a shallow rut along the edge on heavy-traffic roads carrying a continuous double line of traffic (see Fig. 120). This situation, however, applies only to Class I traffic roads and need not be considered in the design of roads, of Classes II, III, and IV, as on these roads there are few cases where heavily loaded wheels follow close to the edge continuously enough to cause trouble.

There is not much probability of adjacent wheel loads of passing rigs being close enough together to cause critical loads continuously in the central part of the road. *Flexible-base pavements on Class I roads must consider the critical location of load. Macadam pavements on roads of Classes II, III, and IV are designed for the normal wheel-load location.*

On rigid pavements the critical location of wheel loads is at the corners of slabs. Such corners occur both at the pavement edge or at internal points due to longitudinal joints or temperature cracks (see Fig. 136, p. 405). Wheel loads at critical points occur quite continuously at internal corners and with moderate frequency at outside corners on heavy-traffic roads. *Rigid-pavement design is predicated on utilizing critical-load location in designing strength.*

Effect of Future Resurfacings on Strength Factor of Safety.—The strength of a macadam road can be increased at any time by the construction of a new top course on the old road. The distributing power of macadam varies about as the square of the depth, and it makes no difference whether the different layers are constructed at the same time or at intervals. This means that if at any time the loading on a macadam road increases or if the original construction proves too weak for increasing traffic the pavement can be strengthened without much loss in total final cost. This warrants using a low factor of safety in original strength design.

While the resurfacing of rigid slabs adds something to their strength, the construction of successive layers is not an effective way to increase beam or slab strength. Rigid slabs must be originally constructed at least amply strong for existing traffic and some provision may well be made for heavier future traffic; that is, the factor of safety should be somewhat higher than for macadam design.

Effect of Construction Imperfections.—The strength of pavements depends largely on the excellence of construction, and in designing some allowance must be made for common imperfections.

Macadam is less damaged by careless manipulation than rigid pavements. The settlement of new fills causes more cracking of rigid pavements than any other factor.

TABLE 64.—SUMMARY OF FUNDAMENTAL FACTORS IN DESIGN

Macadam Types	*Rigid Types*
1. Soil support is utilized at all points.	1. Soil support cannot be utilized at all points.
2. Full soil support can be utilized.	2. Full soil support cannot be utilized.
3. The soil support used must be lowest value, considering seasonal variation.	3. Seasonal variation of soil supporting power has only a minor effect.
4. Macadam strength varies at different times.	4. Rigid slab strength is constant.
5. Occasional excessive loads need not be considered.	5. Occasional excessive loads must be considered.

TABLE 64.—(*Continued*).

Macadam Types	*Rigid Types*
6. The special location of loads need be considered in only a few cases.	6. The special location of loads must always be considered.
7. Temperature stresses need not be considered.	7. Temperature stresses must be considered.
8. A comparatively low factor of safety is warranted.	8. A fairly liberal factor of safety is necessary.
	9. Rigid pavements composed of separate concrete base and resilient surface course-es reduce impact damage and warping due to daily temperature fluctuations and increase wheel-load distribution over base cracks.

If the design of either macadam or rigid pavements provided for the most extreme combination of unfavorable conditions for every factor involved, the pavement would be prohibitive in cost. It is apparently sound policy to provide for probable combinations of unfavorable conditions and plan on minor repair for a small percentage of failure. Reasonable design hinges on the assignment of proper values for the various factors of wheel load, supporting power of soils, distributing power of macadam pavements, and slab strength for rigid pavements.

Traffic Loads.—The maximum loading on pavements is produced by the modern commercial truck traveling at high speed. Horse-drawn farm vehicles and the ordinary passenger automobile can be disregarded as far as the load factor is concerned. Even military field ordinance loads are no more severe than the commercial truck loading.

Statutory limitations on load, speed, and various other factors are necessary to insure against the needless destruction of pavements. Existing statutes limit vehicle gross load, axle load, wheel load, load per lineal inch of tire width, and speed. Such statutes may well be broadened to include limitation of the use of heavy trucks to roads of Classes I and II and also various factors in truck operation which increase impact, such as badly worn tires, spring equipment, etc. While local statutes vary, the essential features have been standardized by common agreement to approximately the following values (1923 for commercial highways):

Maximum gross weight, 22,000 to 28,000 lb.

Maximum wheel load, 8000 to 10,000 lb. (weight at rest).

Maximum load per inch width of tire, 600 to 800 lb.

TABLE 65.—LIMITATIONS OF TRUCK LOADS ON HIGHWAYS (1923)

State	Maximum Gross Weight in Pounds (Four Wheels)
Alabama	20,000
California	30,000
Colorado	16,000
Connecticut	25,000
Delaware	22,000
Florida	16,000
Indiana	24,000
Illinois	32,000
Iowa	20,000
Maine	20,000
Maryland	20,000
Minnesota	28,000
Nevada	25,000
New Hampshire	20,000
New Mexico	12,000
New York	28,000
North Carolina	15,000
Ohio	20,000
Oregon	22,000
Pennsylvania	26,000
South Dakota	20,000
Tennessee	20,000
Texas	22,000
Utah	20,000
Vermont	12,500
Virginia	20,000
Washington	24,000
Wyoming	25,000

The Pennsylvania (1921) law is representative of the usual solid-tire truck-speed regulations.

TABLE 66

Class	Gross load, pounds	Speed, miles per hour
AA	7,000	20
A	11,000	20
B	15,000	18
C	20,000	15
D	24,000	15
E	26,000	12
F	26,000	10

These legal load limitations do not represent design loads, as impact increases the static load of a truck at rest. For well-maintained, moderately smooth pavements, however, the effect of impact does not probably add over 25% to the static wheel load at rest for the larger trucks traveling at legal speeds (see Table 67, U.S. Bureau of Public Roads Impact Tests). For the smaller solid-tire trucks at higher speeds it may add 100%. For pneumatic-tire trucks it does not probably exceed 10% on the average. The tests recorded some cases of 400 to 500% increase due to impact, but these represented conditions of roughness that would not occur

TABLE 67.—IMPACT CAUSED BY DIFFERENT MOTOR TRUCKS
(*U. S. Bureau of Public Roads*)

Rated capacity of truck, tons	Load, tons	Load on one rear wheel			Equivalent static load for different heights of fall		
		Total	Sprung	Unsprung	0	¼″[a]	1″
1½	1½	3,475	2,410	1,065	5,500	7,800	10,200
1½	1½	3,475	2,410	1,065	4,500	6,300	11,500
1½	2¼	4,240	3,175	1,065	4,700	10,000	16,000
1½	2¼	4,240	3,175	1,065	4,900	5,900	9,200
2	2	4,300	3,300	1,000	6,800	7,800	11,500
2	2	4,300	3,300	1,000	6,800	8,500	13,400
2	2	4,300	3,300	1,000	6,800	7,000	11,000
2	2	4,300	3,300	1,000	6,800	7,500	11,000
2	3	4,900	3,900	1,000	7,200	8,500	14,000
2	3	4,900	3,900	1,000	7,300	8,600	13,700
2	3	4,900	3,900	1,000	6,800	8,000	11,000
3-3½	2½	5,150	3,450	1,700	8,400	9,200	15,700
3-3½	2½	5,150	3,450	1,700	8,200	8,700	11,800
3-3½	2½	5,150	3,450	1,700	8,200	9,300	14,100
3-3½	2½	5,150	3,450	1,700	6,900	9,200	21,500
3-3½	2½	5,150	3,450	1,700	8,000	8,200	12,200
3-3½	2½	5,150	3,450	1,700	8,700	8,200	13,000
3-3½	4½	7,000	5,300	1,700	......	10,000	19,000
3-3½	4½	7,000	5,300	1,700	9,700	9,700	19,000
3-3½	4½	7,000	5,300	1,700	9,300	9,800	17,700
3-3½	4½	7,000	5,300	1,700	9,200	11,000	17,200
3-3½	4½	7,000	5,300	1,700	10,000	10,000	18,000
3-3½	4½	7,000	5,300	1,700	10,000	11,000	15,800
5	5	7,900	5,950	1,950	11,000	12,800	18,000
5	5	7,900	5,950	1,950	11,000	12,500	19,000
5	5	7,900	5,950	1,950	9,000	10,000	16,000
5	5	7,900	5,950	1,950	9,200	10,700	15,500
5	5	7,900	5,950	1,950	9,200	9,700	13,000
5	5	7,900	5,950	1,950	9,500	......	15,000
5	7½	10,600	8,650	1,950	12,200	15,300	26,000
5	7½	10,600	8,650	1,950	12,000	13,700	21,000
5	7½	10,600	8,650	1,950	11,500	12,500	18,500
5	7½	10,600	8,650	1,950	11,500	12,600	17,200
5	7½	10,600	8,650	1,950	11,000	11,200	16,800
5	7½	10,600	8,650	1,950	11,200	12,100	15,800

[a] The ¼″ column is assumed to represent probable impact on well-maintained roads.

on well-kept pavements. The extreme loads resulting from impact on rough surfaces emphasize the necessity for the construction of reasonably smooth pavements properly maintained. *It would be impracticable to design pavements on the basis of high-impact stresses.* Attention should be concentrated on the reduction of impact to the minimum feasible amount, which is assumed to be represented by the ¼″ column of Table 67 for the smoother pavements, such as modern macadams, sheet asphalt, grouted brick, and concrete, and a somewhat higher value for the rougher types, such as stone block, brick with bituminous filler, etc. Vialog records (discussed on p. 545) indicate that the ¼″ impact is a reasonable basis of design.

The tractive effort of mechanically propelled vehicles is a surface shearing force and can be disregarded in deriving design loads for pavement-strength formulas. It has some effect in producing humps and hollows in asphaltic-concrete and bituminous-macadam surfaces and probably hastens disintegration of bond between poorly bound block surfaces and is a contributary cause of vibratory disintegration of rigid monolithic pavements.

Recommended Design Loads.—It seems rational to recognize at least two load classifications due to general character of traffic and also variations due to pavement surfaces.

Main Roads of Classes I and II.—These roads should be designed for the heaviest commercial trucks permitted by law. It is probable that a design wheel load of 14,000 lb. (11,000 lb. static plus 3000 lb. impact) distributed over a 12″ tire width represents about the maximum load produced by present-day heavy traffic operating at legal speeds on well-maintained rigid pavements, such as cement concrete, monolothic, or semimonolithic brick with cement-grout joints on concrete base, etc. Pavements having a surface or base which has some flexibility and resiliency are undoubtedly less damaged by impact than the more rigid types. How much weight can be given to this in design is not known, but in order to recognize this factor it is well to adopt a slightly lower value than shown in Table 67. Twelve thousand pounds on a 12″ tire will be assumed for bituminous macadams, small blocks with asphaltic joints on macadam bases, and asphaltic-concrete surfaces on either macadam or concrete bases. For the rougher pavements, such as stone block or brick with bituminous joints on concrete bases, a value of 15,000 to 16,000 lb. is probably not excessive.

Local Roads of Classes III and IV (Macadam Design).—For these roads a lower design load is logical. A total of 9000 lb. (5000 lb. static plus 4000 lb. impact) on a 10″ tire is liberal and a design allowance of 7000 lb. on a 10″ tire width is probably about the desirable maximum limit for purely local roads carrying less than 300 vehicles daily. Roads of classes III and IV are at times subjected to heavy-truck Class I loading under existing traffic-regulation statutes, but it is probably sound policy to use the lighter design loading and make an effort to modify existing regulation to the extent of keeping the heavy trucks off from these local roads, particularly in the spring. While it is probably impossible to prevent heavy trucks from violating such statutes entirely, an occasional violation is not serious, as is noted under General Basis for Design of Macadam Roads (p. 368). The recommended loading permits all-year-round use of the ordinary 2½-ton agricultural trucks and gives sufficient strength in the summer season to permit the use of these roads as temporary detours for heavy traffic at times when the main roads are closed on account of construction or repair.

TABLE 68.—RECOMMENDED DESIGN LOADS

Class of road	Design static wheel load (including impact)
I and II	16,000 lb. on 12″ tire, rougher types rigid pavements 14,000 lb. on 12″ tire, smooth, rigid pavements 12,000 lb. on 12″ tire, resilient pavements
III	12,000 lb. on 12″ tire, rigid pavements 9,000 lb. on 10″ tire width, macadam pavements
IV	7,000 lb. on 10″ tire width, macadam pavements

These concentrated wheel loads must be transmitted by the pavement and applied to the underlying earth subgrade over a large enough area to prevent rapid unequal settlement or permanent displacement.

Supporting Power of Soils.—Different soils have a wide range in supporting power, and even the same soil varies greatly at different times, due to moisture content, freezing action, and degree of consolidation. Road foundation soils fluctuate more in their supporting power than deep foundation soils for buildings, as they are near the surface and are subject to the effects of rain, frost, and the churning action of rapidly fluctuating loads.

Pavement design, to be rational and fairly consistent, must consider variations in the supporting power of the underlying subgrade, but it can only consider such differences in soil conditions as are clearly defined. It is impracticable to give much attention to minor variations; that is, for a specific road, the supporting power of soil varies foot by foot, and even for the same point it changes at different times in the year. For these reasons, any attempt to apply laboratory methods to rapidly fluctuating subgrade conditions is impracticable, to say the least. It is feasible to provide changes for well-defined differences in soil conditions which become evident to an experienced highway engineer during the progress of the survey and construction. It is practicable to eliminate excessive ground-water seepage by the use of underdrains and to reduce surface-water seepage by well-constructed and well-maintained shoulders and ditches and by the constant sealing of cracks in rigid pavements. Even with these precautions, water will work in from the shoulders or reach the subgrade through cracks in the pavement and in this way temporarily reduce the supporting power of the earth subgrade. All pavement and foundation design must be predicated on the fact that the supporting power of the underlying earth will vary at different times, and the important factor in design is minimum supporting power, considering all-year-round conditions for local climatic conditions.

Experience has demonstrated that coarse sands and gravel do not vary much in the supporting power at different seasons if protected by well-designed drainage. It is known that loams, clays, and fine sands vary considerably in supporting power where they are located in cuts or thin "pancake" fills even when protected by well-designed drainage, and that these same soils in fills over 3′ deep do not vary much at different seasons, provided the pavement surface is reasonably waterproof and the shoulders are maintained to shed water rapidly. Figure 121 shows in a general way the effect of moisture content on the supporting power of different soils. Experi-

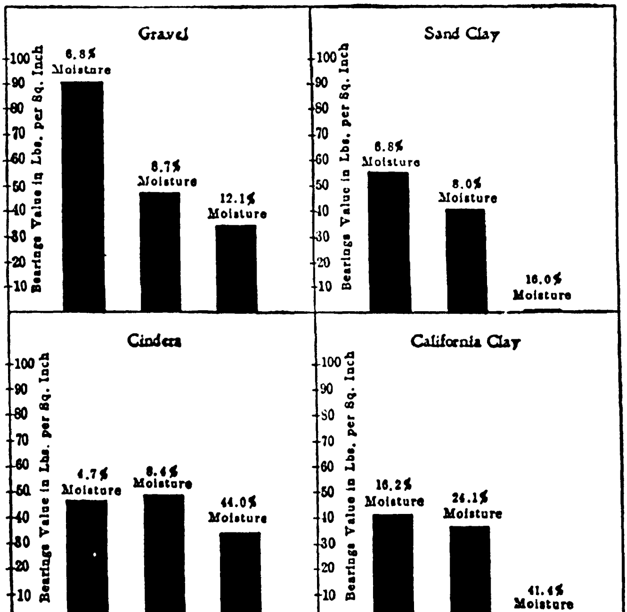


FIG. 121.—Effect of moisture content on bearing power of soils. U. S. Bureau of Roads testing records.

ence has demonstrated that under normal road conditions it is fairly easy to prevent a large variation in the moisture content of coarse sand and gravels and increasingly difficult to control the moisture contents of loams, clays, and quicksands. The vertical rise of moisture due to capillary action is shown in Table 69. Moisture content for roadbeds in cuts is reduced by open ditches and underdrains, but these methods fail in effectiveness at times due to all sorts of reasons, such as frozen drains and shoulders, and poorly maintained shoulders, permitting pools of water to stand and seep in cracks in rigid pavements. Where the road is in cuts or on thin fills they are certain to get at times a higher moisture content than where the roadbed is on a high fill which is free from groundwater seepage and above the limits of vertical capillary lifting action. *As macadam design utilizes full support, it must not only recognize the difference in soils but it must also recognize whether the soil is in cuts or fills.*[1]

TABLE 69.—TABLE OF HEIGHTS TO WHICH CAPILLARY WATER RISES IN DIFFERENT SOILS (*Experiments by U.S. Bureau of Public Roads*)

Soil	Height in 24 hr., inches	Approximate final height, inches
Light sandy soil	14	28
Gravelly soil	16	32
Decomposed granite (loam)	21	40
Heavy granite loam	16	32
Clay loam	12	24
Pure coarse sand	9	18

NOTE.—The percentage of moisture due to capillarity decreases rapidly with the height above the free water for the coarse sandy soils, while it remains nearly constant for the entire range of vertical capillarity in some of the heavier soils.

There seems to be fair agreement between widely different sources as to the minimum safe supporting power of well-compacted subgrade soils protected by well-designed drainage provisions. The Massachusetts Commission in 1901 determined these values at 25 lb. per square inch for coarse sand and gravel and at 4 lb. per square inch for the poorer clays and excessively fine sands. French experiments, quoted in Agg's "Construction of Roads and Pavements," place the values at 30 lb. per square inch for firm soils and at 9 lb. per square inch or less for the poor soils. These values are for well-compacted soils and refer to well-rolled subgrade soils kept free from an excessive moisture content by a well-designed drainage system. The necessity for thorough rolling of the subgrade is well recognized. Table 70 compares these values with the usual value assigned to these soils for deep building foundations and illustrates the radically different supporting value of surface and subsurface soils.

[1] The fact that fills reduce required depth of macadam pavements does not affect grade-line design, as the reduction in cost of macadam rarely balances the increase in grading cost due to arbitrarily raising the road by fill; that is, the consideration of pavement depth and strength never raises the road onto fills except across low, wet, swampy areas. Needless fills add danger to road traffic, always increase damage to abutting properties, and are to be avoided as much as possible, as they give no adequate return. Where fills are necessary, their construction should be taken advantage of to reduce macadam depths but not rigid-pavement depth. Most rigid-pavement troubles occur on fills due to incomplete compaction before the pavement is constructed.

TABLE 70.—COMPARISON OF SAFE SUPPORTING VALUE OF SURFACE AND SUBSURFACE SOILS, IN POUNDS PER SQUARE INCH

Soil	Road subgrade surface soils	Subsurface building foundations
Coarse sand and gravel	25–30	60
Poor damp clay and fine shifty sand (30% or more passing 100 sieve)	4–8	15

The values of intermediate soils and the effect of fills are largely matters of judgment and experience. The following recommended values are based on the observed action of roads in western New York during the past 15 years. These values are to be used in conjunction with Formula 1 (p. 382), and are safe values for localities subject to severe winters. They can undoubtedly be increased for locations free from frost; no special values are given for such conditions, as the author's personal experience does not warrant assigning values except in climates similar to the northeastern Atlantic states. In the course of time, values for various localities will become more or less standardized. With this in view, it is desirable for the various highway departments to make a systematic effort to record failures and successes, as shown in chart form (Fig. 131, p. 388). One of the difficulties to overcome in public highway work is the effect of rapidly changing personnel. If data are not systematically recorded, the value of experience is lost and the new men have to start in all over again to gather data. Up to date, this has been one of the main difficulties.

It can be readily seen that the classification of the soils is largely a matter of judgment, but it is entirely feasible to educate a begin-

TABLE 71.—RECOMMENDED SAFE SUPPORTING POWER OF SUBGRADE SOILS, IN POUNDS PER SQUARE INCH, FOR LOCALITIES SUBJECT TO SEVERE WINTERS

(For use in connection with Formula 1, p. 382.)

Soil	Location		
	In cuts or on fills less than 1′ deep, pounds per square inch	Intermediate fills, 1 to 3′ deep, pounds per square inch	High fills, over 3′ deep, pounds per square inch
Fine sand (more than 30% passing 100 sieve)	4– 8	8–15	15
Heavy clay	4– 8	10–17	17
Ordinary clay	8–13	13–19	19
Clay loam	13–17	17–20	20
Loam	17–21	19–21	21
Sandy loam	21–25	23–25	25
Coarse sand	25–30	27–30	30
Fine gravel	25–30	27–30	30

ner very rapidly to distinguish between the general classifications noted in Table 71. Any attempt to classify subgrade soils during construction by laboratory methods is too costly and complicated a process to have much practical value. If the design of macadam is based on the general classifications given, the chances are all in favor of reasonably good results, particularly if the grading operations are controlled and the fills under the pavement made fairly uniform and of the best available material.

STRENGTH DESIGN, FLEXIBLE-BASE PAVEMENTS

Theoretical Design of Macadam Road Thickness.—The determination of thickness depends on the wheel loads, the supporting power of the underlying soil, and the distributing action of the macadam pavement. Wheel loads and the supporting power of soils have been determined within reasonable limits of accuracy. These determinations are based both on experiment and on the observed action of macadam pavements under actual traffic, which indicates quite strongly that they are essentially correct. It remains to discuss the distributing power of macadam pavements and to assemble the factors with a reasonable factor of safety.

Distributing Action of Macadam, Gravel, and Boulder Bases.—Experimental data in regard to the load distribution through flexible-type bases are meager. Table 72 taken from Agg's "Construction of Roads and Pavements," records certain French data which are probably as reliable as any investigations which have been made up to this time (1922). Figures 122 and 123 (p. 379) show recent tests made by the U.S. Bureau of Public Roads at Washington.

TABLE 72.—SHOWING TRANSMISSION OF PRESSURE THROUGH MACADAM

(The test was made with a wheel load of 4 tons with a $5\frac{1}{2}''$ tire)

On macadam alone:					
Thickness of crust, inches	1.97	3.94	5.91	7.87	11.81
Pressure on subsoil, pounds per square inch	102.50	47.70	27.40	17.40	9.10
On Telford foudation alone:					
Foundation thickness, inches	......	.....	5.91	7.81	11.81
Pressure on subsoil, pounds per square inch	......	.....	56.00	37.40	20.70
On combined foundation and macadam:					
With foundation thickness of, inches	......	5.91	7.87	9.84	11.81
3.94" of top stone, pressure on subsoil, pounds per square inch	......	19.30	14.70	12.60	10.20
5.91" of top stone, pressure on subsoil, pounds per square inch	......	13.20	10.90	9.10	7.70
7.87" of top stone, pressure on subsoil, pounds per square inch	......	9.70	8.20	6.80	6.10

NOTE.—The pressure on the subsoil through a 12" bed of simple macadam is apparently the same as the pressure through a 6" bed of macadam laid over a 10" stone foundation. The pressure transmitted by well-constructed gravel will be about the same as with macadam of equal thickness.

A good soil well drained will safely withstand a pressure of 30 lb. per square inch. It is considered that poor soil requires at least a 12" macadam layer or its equivalent.

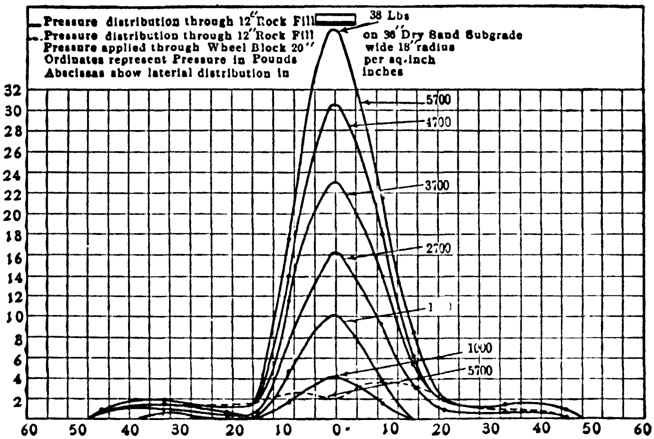


FIG. 122.—Distribution of pressure through macadam laid on sandy soil. U. S. Bureau of Roads experiment.

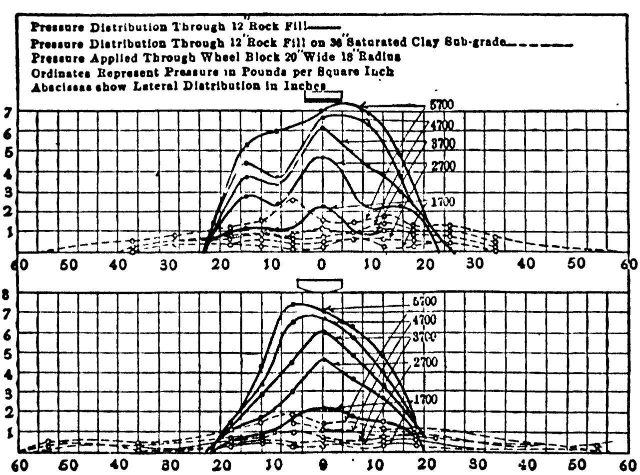


FIG. 123.—Distribution of pressure through macadam pavement clay sub-soil. U. S. Bureau of Roads experiment.

The distributing action of broken-stone macadam is slightly better than Telford or boulder base. The distributing action of well-constructed gravel courses is usually as good if not better than boulder base, although not quite so effective as macadam. The distributing power of all these types depends on the excellence of construction (degree of tight interlock and compaction), and for each type it will vary at different times of the year, the least value occurring in the spring when the tight internal lock is reduced by frost heave. In order to provide the proper thickness of macadam it is necessary to determine the effect of depth on total area of subgrade over which the wheel load is distributed and the ratio of maximum intensity of pressure directly under the wheel to average intensity over the total area of subgrade receiving pressure.

Figures 122 and 123 show that for a given depth of macadam and a certain specified load the maximum intensity of pressure directly under the wheel is greater for firm soils, such as dry sand, than it is for soft subgrades, such as wet clay; that the ratio of maximum pressure to average pressure is greater on firm soils; and that the total area of load distribution is slightly less on firm soils than on soft subgrades. This merely means that firm soils offer greater unit resistance and that, for the condition of a certain specified depth of macadam and a specified load, the deflection of the macadam under the wheel is less on the firm subgrade than on the soft subgrade. These variations in the law of pressure distribution disappear in any rational formulas for macadam design for the following reasons: A well-designed macadam road must not have much deflection under a moving wheel load, for the minute noticeable deflection occurs (so-called churning), the pavement starts to disintegrate, and either gets out of shape rapidly or completely goes to pieces. Some deflection is bound to occur, but the basic principle of good design requires variation in stone depth over different soils, so that the maximum wheel load produces a fairly uniform deflection at all points. This deflection must be small enough to prevent churning. If the road is properly designed with a uniform and safe deflection, the relation of depth to distribution area and the ratio of maximum intensity are not probably much affected by the character of the subsoil. At least it is known from practical experience that simple empirical formulae, which consider variations in wheel load, a fairly well-established relation between depth and intensity of pressure on the subgrade, and reasonable values for safe supporting power of soils, have been developed, and that such formulae have been given sufficiently hard and long enough continuous service tests to warrant their use as a basis of economic macadam design.

An old and simple rule will be used as the basis of a practical formula, namely, the Massachusetts rule of 1901: "The pressure is distributed through macadam and any ordinary gravel or boulder base at an angle of 45° with the horizontal, and the resultant maximum intensity of pressure on the subgrade amounts to the load at any point divided by the square of twice the depth of the pavement." While this is rather a rough-and-ready rule, its application produces as good results as any basis of comparison, provided it is slightly modified to consider width of wheel. Formula (1) makes such minor modifications. A comparison of Formula (1) with the French experiments (Table 72) is shown in graphic form in Fig. 124, and indicates essential agreement as far as practical results are concerned, with the added indication that formula (1) is on the safe side for the usual combinations of macadam top and boulder bases.

Research engineers are not likely to approve heartily of this formula, as it is not a very scientific product, but its simplicity,

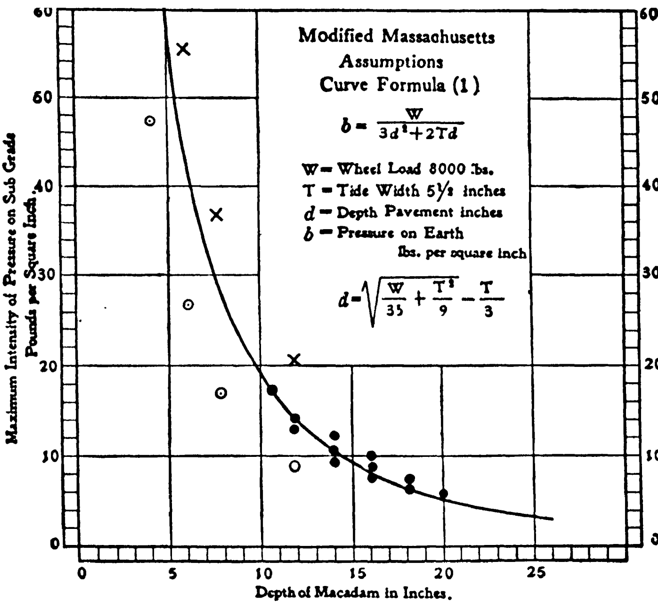


FIG. 124.—Graphic comparison of Formula 1 (solid curve) with French experiments recorded in Table 72 (plotted by crosses, circles and dots).

its essential agreement with experimental data, and the excellent results obtained in actual construction from its use are strong arguments in its favor.

Basic Assumptions (Formula (1)). *Area Uniform Maximum Pressure.*—A hypothetical area of uniform maximum pressure is determined by a line at an angle of 45° from the outer limits of wheel contact. This, of course, does not represent actual conditions, but it is on the safe side. The pressure caused by a wheel load is not uniformly distributed over the subgrade (see Figs. 122 and 123, p. 379). The maximum pressure occurs directly under the wheel and gradually fades away to nothing at a variable distance from the point of load. The assumption used gives the same total resistance to the wheel load, but contracts the actual area of the subgrade receiving variable pressure to a hypothetical area over which the maximum pressure occurring directly under the wheel is considered as uniform. The essential correctness of the 45° limit of uniform maximum pressure can be seen from Figs. 122, 123, and 124. By this means the value for safe supporting power of soils can be used directly.

The area of maximum uniform pressure depends on the area of wheel contact, which depends on width of tire and length of contact along the road. The width of tire is a definite dimension, recognized and limited in statutory load regulations. The length of tire bearing along the surface is variable, depending on size of the wheel and on the type of tire and load. For steel-tired wheels it is small, probably not over an inch. For solid-tired heavy commercial trucks it is a larger value. Since the steel-tire load is liable to occur and the length of contact is variable for rubber tires, this dimension will be neglected and knife-edge contact for the full width of the tire will be used. This gives results on the safe side and, in effect, usually introduces a factor of safety of about 10 to 25%. It is also assumed that the wheel load is far enough from the edge of the pavement to get full distribution for the entire depth of pavement. Modifications for edge loading are considered in Formula (2).

The formulas for depth of macadam are developed on the basis outlined.

W = static wheel load, in pounds plus impact allowance.
T = width of tire, in inches.
d = depth of pavement, in inches.
b = maximum pressure on subgrade due to wheel load, in pounds per square inch.

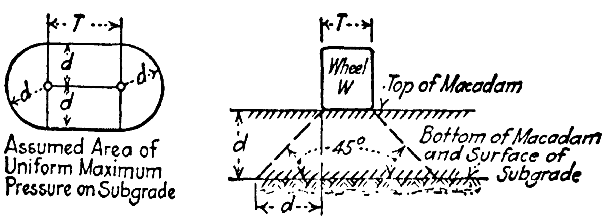


FIG. 125A.

Area of uniform distribution of maximum pressure $= \pi d^2 + 2dT$
$= 3.14\,a^2 + 2dT$
$= 3d^2 + 2dT$ for all practical purposes.

$$\left.\begin{aligned} b &= \frac{W}{3d^2 + 2dT} \\ d &= \sqrt{\frac{W}{3b} + \frac{T^2}{9}} - \frac{T}{3}. \end{aligned}\right\} \qquad (1)$$

Modification of Formula (1) for Special Location of Wheel Loads.—A typical maximum-load condition is shown in Fig. 126 for an 18-ft. double-track road. This is a rare occurrence, as when trucks pass they want all the clearance they can get and they hug the edge of the road (Fig. 127).

These sketches indicate roughly that there is not much probability of enough overlap of pressure zones from adjacent wheels in the center of a road to produce a greater pressure than the maximum directly under the wheel for ordinary depths of macadam on

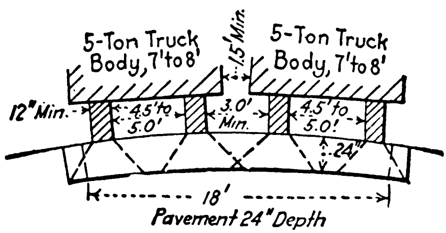


FIG. 126.—Location of passing vehicles for critical load in center of macadam.

ordinary soils; for extreme depth on very poor soils such an overlap may occur, but it would require a very unusual combination of unfavorable conditions to make this continuous enough to cause trouble.

On the side of the road, however, edge loading is very common and continuous on Class I roads, but it occurs only at intervals along the road and only at time intervals on roads of Classes II, III, and IV. This critical condition of side load must be considered

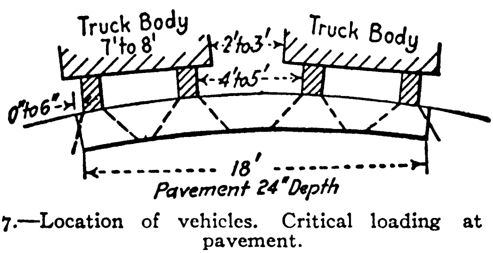


FIG. 127.—Location of vehicles. Critical loading at edge of pavement.

if the macadam type is used on Class I roads. As previously stated, the use of macadam is inadvisable on Class I roads except for special conditions in reconstruction programs. Formula (1) need not be modified for roads of Classes II, III, and IV, but probably should be modified for Class I roads.

Any modification is purely guesswork and is made more to point out and recognize the effect of critical loading on Class I roads than for any other reason. Under the worst possible conditions, eliminating all distributing action by the shoulder material, the supporting area may be assumed as

$$2dT + \frac{\pi d^2}{2}, \text{ say, } 2dT + 1.5d^2.$$

This is probably entirely too low a value, and to approximate conditions and to give some value to shoulder support, the area will be assumed at

$$2dT + 1.5d^2 + \pi\left(\frac{d}{2}\right)^2$$

$$\text{say} = 2.25d^2 + 2dT.$$

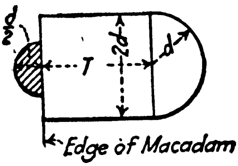


FIG. 128.

Using this value, Formula (1) becomes
For Class I traffic:

$$\left.\begin{aligned} b &= \frac{W}{2.25d^2 + 2dT} \\ d &= \sqrt{\frac{4W}{9b} + \frac{16T^2}{81}} - \frac{4T}{9} \end{aligned}\right\} \qquad (2)$$

For traffic of Classes II, III, and IV:

$$\left.\begin{aligned} b &= \frac{W}{3d^2 + 2dT} \\ d &= \sqrt{\frac{W}{3b} + \frac{T^2}{9}} - \frac{T}{3} \end{aligned}\right\} \qquad (1)$$

Utilization of Formulas (1) and (2) in Macadam-pavement Design.—To utilize these formulas, it is necessary to assemble the values for load, supporting power of soils, and to discuss the factor of safety desired. Load and distributing power of macadam when tight locked are reasonably certain. Live load will vary according to the traffic the road will have, but for all practical purposes it is safe to use the statutory limitation plus an allowance for impact. The values given on page 374 contain a small factor of safety. Dead load of the pavement itself can be safely disregarded except when dealing with muck or swampy foundation. The formulas developed for distribution of pressure assume the same distribution for all the usual combinations of base and top courses and disregard reduction in distributing power due to frost action for a short time in the spring. Figure 124 indicates that for the usual combinations of base and top these formulas are on the safe side, but this factor is probably balanced by the unknown reduction in distributing power during the spring break-up. Safe soil-supporting power is the most indefinite factor in the problem. As this factor is largely a matter of personal judgment, the values assigned for supporting power of soil can provide the desired factor of safety in the result.

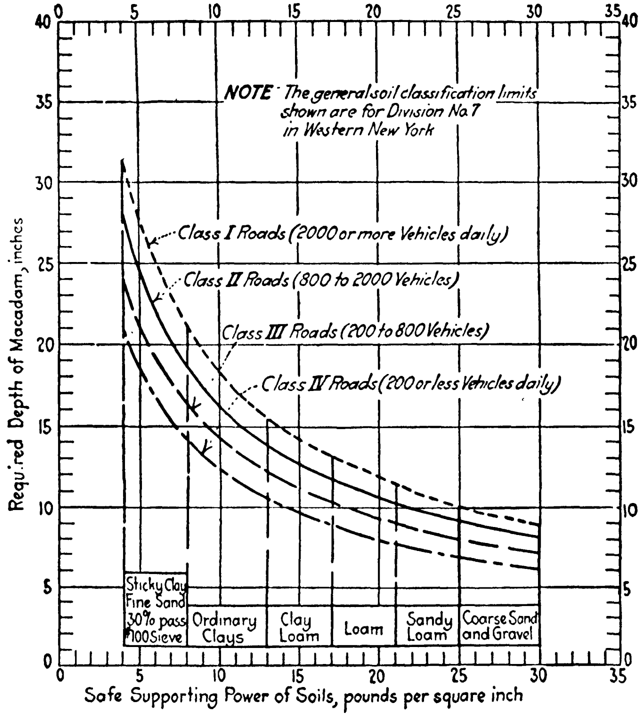
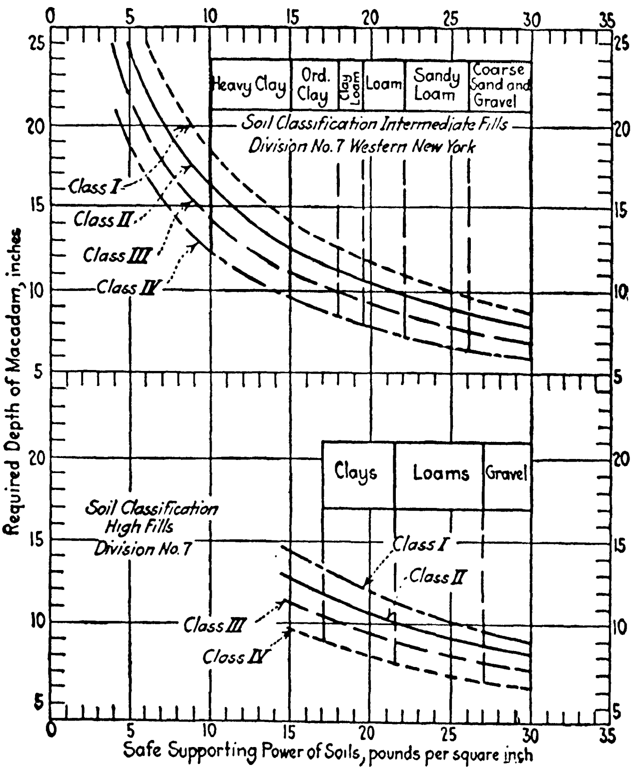


FIG. 129A.—Recommended depths of macadam pavements on different soils where the pavement is located either in cut or on fills less than 1 ft. deep.

The values given on page 377 were adjusted for use in conjunction with this formula; it will be noted that the values used for the extreme conditions of good and bad agree closely with the Massachusetts and French values. The intermediate values are based on experience in western New York for a number of years. The use of these values for districts similar to western New York will give reasonable results. There will be some spring failures, but not many, and for the balance of the year the factor of safety is high.

Figures 129, 130*A* and 130*B* show graphically the depths of macadam obtained by the use of Formulas (1) and (2) with the recommended live loads given on page 374 and the soil values given on page 377.

The suitability of these curves for any district can be easily determined by plotting on a chart the record of actual road failures



FIGS. 130*A* and *B*.—Recommended depths of macadam pavements located on fills. Intermediate fills (1 to 3 ft. deep). High fills (over 3 ft. deep).

for the district in question. If the curve derived from Formula (1) (using the local load limitation) is very much above the average run of actual blow-ups, it can be lowered to get a reasonable percentage of failure; that is, this curve should about fit the upper limit of depth for the usual failure on the different soils. A few exceptional cases can be disregarded. The data of actual road failures should be charted in a similar way to the usual chart of storm intensity in deriving storm-sewer design assumptions. This method is illustrated by Fig. 131 (p. 388), which is a record of observations in western New York, 1907 to 1920.

Formulas (1) and (2) reduce to the following form if the design loading recommended on page 374 is used:

Class I roads:

$$d \text{ (depth macadam)} = \sqrt{\frac{48{,}000}{9b} + 28} - 5.3.$$

Class II roads:

$$d = \sqrt{\frac{12{,}000}{3b} + 16} - 4.$$

Class III roads:

$$d = \sqrt{\frac{9000}{3b} + 11} - 3.3.$$

Class IV roads:

$$d = \sqrt{\frac{7000}{3b} + 11} - 3.3.$$

These formulas are used in the preparation of Figs. 129, 130*A* and 130*B*.

Experience with Various Depths of Macadam in Western New York.—The first roads built in this section (from 1898 to 1902) were quite uniformly 6″ in thickness over all kinds of soils. This was admitted to be rather irrational design, but the cost had to be kept down and it was considered better policy to plan on more or less failure with consequent repair rather than spend more to insure against minor failure. As experience was gained, the depths were varied somewhat over different soils, but these depths were in all cases much less than are used today. The growth in motor traffic with heavier loads and more vehicles has resulted in the failure of most of these old thin macadams, and the old roads have been gradually strengthened by the addition of extra courses of stone to meet the increased loads. By observation of these successive thickenings, the State Department of Highways has had a very excellent opportunity of drawing reasonable conclusions as to the depths required for handling modern traffic on different soils.

There has, however, been no systematic official effort to gather these data in a complete and convincing manner, and what data are available are due to the individual efforts of a few of the engineers. Officially, the experience of 20 years has been thrown away as far as definite tabulated data in regard to economical macadam depths under different conditions go. This is very typical of the attitude towards highway work in many cases, and some executives have even said that they did not want to know too much about the economics of different types of pavements. This general attitude is being gradually discredited, and there are hopes that in time an effort will be made to record the number of square yards of different depths on different soils with the yearly percentages of failure for the various depths and in this way gain some reliable means of balancing original cost against minor repair. This method has been used by various cities in determining economic depth of concrete foundations for street pavements.

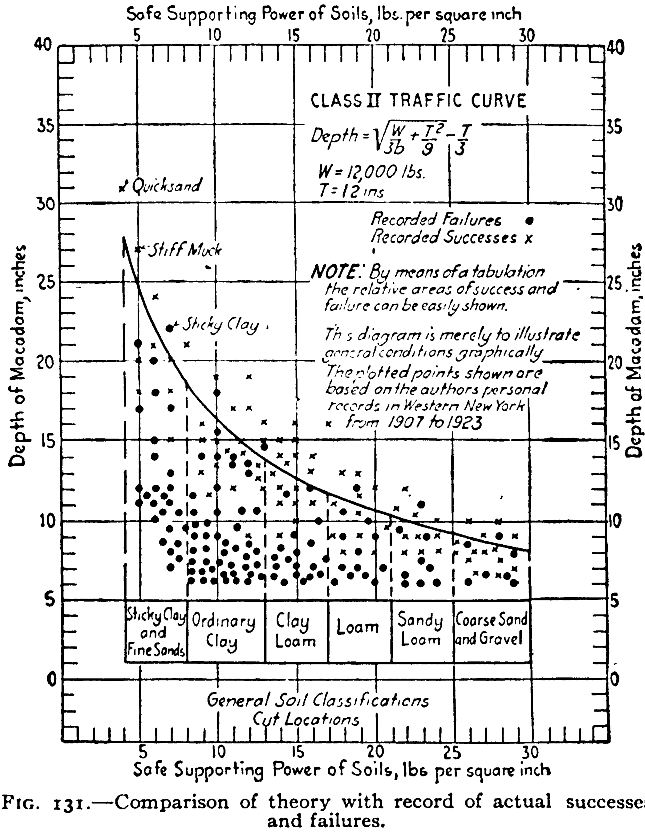


FIG. 131.—Comparison of theory with record of actual successes and failures.

The following statements summarize the experience and judgment of a number of the engineers of Division 4 who have kept records of successes and failures under the test of modern traffic, including army trucking during the war.

"Pavements 22" thick laid on poor clay and stiff muck soils have in a few exceptional cases rutted badly under Class II traffic. Pavements 23 to 27" thick under similar soil and traffic conditions have retained their shape and have required only minor repair. This represents the worst soil conditions encountered in this territory. As a rule, a depth of 20" even on very poor clay holds Class II traffic with only minor failures.[1]

"On coarse sand and fine gravels, we have cases where a 7" depth is serving Class I traffic with no evidence of weakness, but these are exceptional cases. As a rule, 9" is considered about the minimum serviceable depth for Class I on excellent soils.

"For loams, 7 to 8" depths have in some cases lasted well under Class II traffic, but as a rule 10 to 12" is considered necessary to avoid too much maintenance.

"For ordinary clays 9" often lasts quite well under Class II traffic, but enough trouble has occurred even with 12" depth on these soils to warrant 15 to 18" as a reasonable design depth under Class II or III loading."

Current Practice in Macadam Depths (1922).—In order to coordinate this experience with other localities, Table 73 records the range in macadam depths used by other states. Figure 131 shows a graphic method of charting successes and failures.

TABLE 73.—RANGE IN MACADAM-PAVEMENT DEPTHS

State	Minimum depth on firm soils, inches	Maximum depth on poor soils, inches
New York	8 (gravel soil)	12–30 quicksand
Rhode Island	6	9–20
Indiana[a]	11	14 or more
Pennsylvania[a]	9	12 or more
West Virginia[a]	8	12 or more
Arizona	6	Whatever needed
Wisconsin[a]	8	12 or more
California	6	Whatever needed
Washington	7	12 or more

[a] The maximum depths given for these states do not indicate actual extreme maximums for short stretches of exceptionally poor soil conditions.

Effect of Soil and Total Depth on Selection of Foundation Courses for Macadam Pavements.—No matter what total pavement depth is required, the upper 3½ to 9" are usually of the true broken-stone or broken-slag macadam. Below this macadam, either stony gravel, boulder base, or quarry-stone Telford can be used to get the total depth required with the least cost. If quicksand or wet clay is encountered, at least 6" of cinders or gravel should be placed as the lowest layer in order to blanket the soil and prevent its working up into the stone courses. Above this either gravel or boulder base or Telford can be used, capped with the usual broken-stone bottom and top courses. In case gravel subbase is used for the entire depth of subbase, at least 5 to 9" of macadam must be used for the cap, as it is not safe to produce a pressure of over 30 lb. per square inch on fine gravel without danger of rutting, and the object of the overlying macadam is to reduce wheel pressure to this value (see Fig. 130*A* gravel soils). If the gravel subbase is exceptionally coarse stony gravel, it is safe to reduce the macadam cap thickness to 4" for Class IV roads and to

[1] For deep macadams, the upper 3½ to 7" are usually broken-stone macadam with the balance of the depth coarse gravel, boulders, or Telford subbase courses.

8″ for Class I roads. If boulder or Telford base immediately underlies the macadam, the thickness of the macadam can be reduced to 3½ to 5″, as the boulder base has better supporting power than the gravel for direct concentrated loading; that is, the object of macadam cap over the boulder base is to smooth the road for the use of traffic.

The line of pressure distribution is about the same through boulder base or gravel, with the evidence slightly in favor of the gravel.

The use of gravel for subbase does not increase the total necessary pavement depth, but it does increase the necessary thickness of the true macadam type of surfacing (see Figs. 132 and 133).

For any depth up to 8″ the true macadam top and bottom courses are generally selected. For greater depths, any combination can be used, keeping the depth of the macadam capping to the minimum safe depth for the type of subbase used.

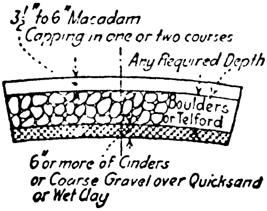


FIG. 132.

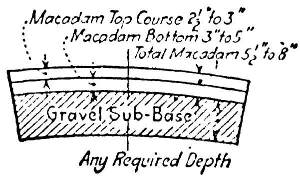


FIG. 133.

FIGS. 132 and 133.—Effect of type of base on thickness of macadam surface.

Recommended Depths.—From the foregoing theoretical and practical data the following recommended depths are derived (Table 74). Such recommendations are intended for use in localities subject to severe winters and for the design loading outlined in Table 68 (p. 374). They are intended as a guide only, to indicate the trend of design practice. There is no intention of creating the impression that they are necessarily the best solutions for all cases, but they can be used with assurance of reasonable success in case local practice has not been crystallized by long experience.

The foregoing discussion shows that, while in a strict sense macadam-pavement depth is not susceptible to true engineering design, it is susceptible to a rational analysis which tends to produce better-balanced results than the purely hit-or-miss methods that have been more or less in use for the past 15 years. *The flexible type of pavement, is generally the most economical type for 90% or more of the road mileage in most localities, and for this reason rational design is essential.*

Asphaltic-concrete and Block Surfacing on Macadam Bases.—This type of road is a desirable design where an old, firm, well-consolidated macadam road needs a better type of surface. *Asphaltic concrete or any high-type block surface should never be laid on new*

TABLE 74.—RECOMMENDED TOTAL DEPTHS OF FLEXIBLE PAVEMENTS IN LOCALITIES SUBJECT TO SEVERE WINTERS (IN INCHES)

Soil	Location of road: In cuts or on shallow fills less than 1′ deep	On intermediate fils 1 to 3′ deep	On high fills over 3′ deep
Class I traffic (over 2000 vehicles daily, 10-hr. count in summer) (Macadam not usually economical on Class I roads)			
Coarse sand and fine gravel	9–10	9–10	9
Loams....	10–14	10–12	10
Ordinary clays.........	15–21	12–16	11
Heavy clays and fine sands	22–30	14–18	13
Class II traffic (800 to 2000 vehicles daily, 10–hr. count in summer)			
Sand and gravel..........	8–9	8–9	8
Loams................ ..	9–12	9–11	9
Ordinary clays...........	12–18	11–15	10
Heavy clays and fine sands	18–28	13–16	12
Class III traffic (300 to 800 vehicles daily, 10–hr. count in summer)			
Sand and gravel.........	7–8	7–8	7
Loams..................	8–10	8–9	8
Ordinary clays...........	12–16	9–14	9
Heavy clays and fine sands	18–24	12–15	11
Class IV traffic (less than 300 vehicles daily)			
Sand and gravel..........	6–7	6–7	6
Loams..................	7–9	7–8	7
Ordinary clays...........	10–15	8–12	8
Heavy clays and fine sands	15–22	10–14	9

macadam, as it is impossible to get complete consolidation of macadam without the added help of traffic pounding.

Firm macadam bases having the thicknesses previously discussed make an ideal foundation for asphaltic concretes and block with mastic fillers, as temperature cracks are largely eliminated. Temperature cracks are the main difficulty encountered where these pavements are laid on concrete foundations. Very excellent results have been obtained in Rochester, N. Y., where this type of construction has been used extensively.

The general conclusion to be drawn is that where an old, firm macadam of adequate depth has been constructed in the past it makes an excellent base for a higher-type surfacing. For the quick construction of an asphalt or brick surface on a heavy-traffic unimproved road the concrete base is the most feasible design. It is well to bear in mind, however, that, while from a practical standpoint the use of a concrete base on a new grading is the best solution for quick results, expensive failures often result from the construction of concrete on new fills or over sewer trenches that have not had time to settle; it is desirable to wait at least a season after a road is graded before placing a rigid pavement. This delay is rarely possible, but if it is feasible the macadam-base type has the advantage, as the macadam base gives good service to traffic while it is consolidating and eventually produces a more satisfactory base than concrete except for exceptionally heavy volume and unit traffic (Class I roads). Where asphaltic concrete is used on top of macadam, the depth of asphaltic surface can be considered as equivalent to an equal depth of macadam. On this basis Table 74 is directly available for computing required total depths.

STRENGTH DESIGN, RIGID PAVEMENTS

Rigid-pavement Design.—Rigid pavements are more difficult to analyze for strength design than the macadam type, but an approximation can be derived. The following discussion is not a scientific analysis in any sense of the word, as this is impossible at the present stage of experimental knowledge. It is desirable, however, to develop empirical formulas which are based on available experimental data coordinated with actual traffic tests.

Considerable progress has been made in the last few years towards a fairly well-balanced design of strength. Investigations in regard to temperature warping, fatigue of materials, soil support, etc. have established fairly definite general principles which have modified the old-style design to advantage. These modifications include reduction in unit width of monolithic-type slabs, increase in the strength of the outer edge of narrow rural highway pavements, and improvement in quality and manipulation of materials. Theory developed from experimental research seems to have been essentially verified by traffic tests, and while there is considerable range for judgment in assigning values to the various factors in the formulas, there is no serious disagreement in regard to essentials. *A discussion of formulated design is given with the idea of bringing out the factors of the problem for the student rather than of providing hard-and-fast standards of designs.* As a matter of fact, a careful analysis of any theoretical discussion of pavement design shows plainly that experience with actual traffic tests is the most reliable basis for action. Table 86 (p. 426) gives the summarized recommendations based on both theory and practice. This table is of definite practical value.

Design Fundamentals (Rigid Slabs).—As outlined on page 366, rigid pavements are designed to bridge over small areas where there is either partial or complete loss of soil support. If large areas of the subgrade settle materially, the pavement fails; it is necessary to provide enough slab strength to bridge all areas of poor soil support which ordinarily occur under the usual construction conditions, but the cost of designing a pavement for exceptional areas of settlement is prohibitive.[1] It is, of course, impossible to determine exactly what size these commonly occurring weak areas will be, and it is necessary to assume some maximum condition of loss of soil support for which provision will be made. The so-called corner-load formula, developed in 1919 by the Illinois Highway Department, Clifford Older, Chief Engineer, seems to afford a practical mathematical basis of comparison of slab strengths ordinarily needed to produce economical pavements which will not have too great percentages of weak areas. *This formula should, however, be used as an index of relative strength rather than as a basis of absolute strength, as it eliminates soil support as a direct factor in the computations.*

Observed pavement failures indicate the predominance of corner-area weakness. Recent experiment has determined that soil contact under corners and along the edges of slabs is quite commonly completely lost, or at least reduced to negligible value. This condition is due to temperature warping, to compression of subgrade due to edge deflection of slabs under heavy traffic, and to increase in the moisture content of the subgrade soil by ground-water seepage and the seepage of surface water through cracks and down through ruts along the edge of the pavement.[2] The extent and the frequency of serious loss of soil support can be reduced by narrowing the width of slabs, by constant sealing of surface cracks, by well-constructed and maintained shoulders, and by effective underdrainage, but even with the greatest care in these particulars soil support is unreliable at corners. These recognized conditions led to the development of the corner-load formula. It is expressed as

$$d = \sqrt{\frac{JW}{S}},$$

in which d = depth of slab, in inches.
W = design wheel load, in pounds.
S = maximum allowable design tension value, in pounds per square inch.
J = variable factor determined by load distribution and location of critical load.

Soil support is disregarded as a direct factor in the computations.

While all the factors W, S, and J are more or less indefinite, actual road service tests are rapidly eliminating any wide range of uncertainty. The formula can be used directly for most cases with good results, but it has more real value as an index of relative strength than as a basis of design of absolute strength on account of complete disregard of soil support. The results obtained by the formula are susceptible to percentage modifications due to reasonable allowances for normal conditions of some soil support. Such modifications depend on direct traffic tests for normal conditions. The impossibility of exactly determining the value of soil

[1] Some efforts have been made to develop an economical design on the basis of pillar support extending below frost line. This basis of design simplifies the design stress problem, but it is not likely that any system of bridge-floor slab design can compete economically with a base partially in contact with the earth subgrade.

[2] "Bates Experimental Road" *Bulletins*, Illinois Department of Highways.

support is the stumbling block in applying scientific design to any type of pavement, either rigid or flexible. The formula is developed as follows:

The Corner-load Formula.—Rigid pavements are considered as a series of slabs. This is essentially correct, for most modern monolithic pavements are constructed with well-defined joints, and most two-course pavements which may be originally constructed as a continuous slab are later broken up by temperature and traffic action into a series of slabs. Mr. Older is quoted as follows:

"*Corners Critical Points.*—It may be safely stated that cracks and joints, *if properly cared for*, affect the life of a rigid slab only as they may affect its strength or load-carrying capacity.

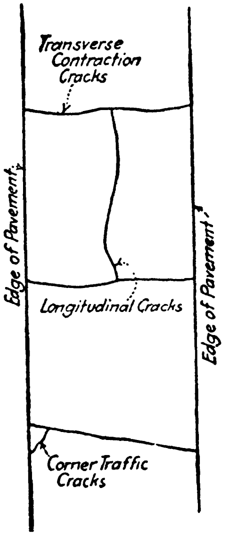


FIG. 134.—Typical cracks.

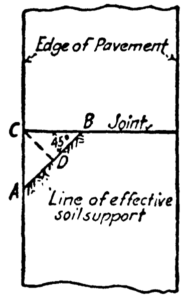


FIG. 134*A*.—Corner load formula diagram.

"It is not necessary to apply mathematical calculations to develop the fact that a load applied near the center of a comparatively large unbroken pavement slab would produce much lower stresses therein than would be produced were the same load applied at or near the edge of the same slab. Further, it is evident that a load applied at the edge of an unbroken slab would not produce stresses as great as those produced by the same load placed at a corner formed by cracks or joints. It is obvious that these are bending stresses which must be withstood by the resisting moment of the slab.

"Regardless of the supporting capacity of the subgrade, it is the corners formed by intersection of cracks or joints with each other and with the edges of the pavement that constitute the weak points of rigid slab pavements. On the Illinois roads which carry truck traffic many broken corners have been observed in all types of rigid pavements, and only in a single instance has a traffic break been observed that was not readily traceable to the piecemeal breaking down of corners or narrow strips formed by longitudinal cracks making acute angles with the edge of the slab. The exception was an asphalt top pavement on a 1:3:5 concrete base 4" thick at the sides and 5" thick in the center, which broke into small pieces under excessive truck traffic. An examination of the base showed that its transverse strength had been exceeded even in the center of the slab. No concrete slab 6" or more in thickness has broken except at corners or except over trenches or other causes of localized settlement.

"Until we are able to control completely the cracking of rigid slabs it seems obvious that we must design the entire slab to carry the imposed loads at the weak point—*the corners.*"

The last paragraph of Mr. Older's statement must not be construed to mean that pavements must be of uniform thickness throughout, as this would result in unbalanced and uneconomical design; that is, the location of corners (such as along the edge of the pavement as compared with interior corners) and other factors later discussed have considerable bearing on the stress produced by normal wheel loads. If due allowance is made for the location and special conditions of load application at different kinds of corners, this test (corner resisting moment) can be used quite successfully in the design of pavement strength.

The relation of traffic load to necessary corner resisting moment of the slab is quite simple, assuming complete loss of soil support for a small area under the corner, load applied at extreme corner, intersection angle of joints, sides, and cracks 90°.

The resisting moment of the slab along the line AB (Fig. 134A) is

$$R = \frac{SI}{\frac{d}{2}} = \frac{Sbd^3}{6d} = \frac{Sbd^2}{6},$$

where R = resisting moment of slab.
S = allowable tensile stress in outer fiber.
I = moment of inertia of the section along line $AB = \frac{bd^3}{12}$.
d = depth of slab, in inches.
b = distance AB, in inches.

The load moment is expressed as load L in pounds, times the moment arm CD in inches. At right-angle corners the distance CD is always one-half of $AB = \frac{b}{2}$ no matter what distance CD becomes. This gives a constant relation between the load moment and resisting moment, regardless of variations in the size of areas of no support under corners.

$$\text{Load moment } M = \frac{Lb}{2}.$$

The load moment and the resisting moment must be the same where soil support is completely lost.

$$\frac{Lb}{2} = \frac{Sbd^2}{6}$$

$$d^2 = \frac{3L}{S}$$

$$d = \sqrt{\frac{3L}{S}}.$$

For convenience, this is modified to $d = \sqrt{\frac{JW}{S}}$ where W equals the design wheel load (discussed on p. 374), J equals 3 × the decimal part of W, which can be considered as applied at the extreme corner, and S equals the maximum allowable design tension value for the material or combination of materials making up the road slab.

Utilization of Corner-load Formula in Design.—The successful use of this formula depends on the determination of reasonable values for tensile strength of the slab S, wheel load W, load distribution at joints in conjunction with location of load J, and soil-support modifications which control the factor of safety of the finally adopted depth. A short discussion of each factor follows. They will then be assembled for the different types of pavement and a table prepared which shows roughly the effect of load on depth and compares the depths required for the different types of pavement under specified legal load limits.

Design Values for Tensile Strength S.—Pavement slabs are constructed of plain concrete without any reinforcement, concrete containing small percentages of imbedded steel, combinations of cement concrete and brick with a fairly firm bond (so-called monolithic brick), and combinations of bituminous concrete or block surfaces on cement-concrete bases with a well-defined plane of weakness between base and surface courses. All pavement slabs subject to bending stresses fail from tension weakness. It is well established that it is uneconomical and impracticable to attempt to reinforce thin pavement bases with enough steel to take full advantage of the compressive strength of concrete. The tensile resistance of concrete can be raised slightly for small areas, such as corners, by means of bar reinforcement. Construction imperfections in mix and materials can be equalized by light mesh, which acts as a fibrous tie, and the use of small amounts of steel in this way permits a rise in allowable design stress. If failure occurs it is always a tension failure. It is therefore necessary to consider only the safe tensile strength of the materials or combination of materials in computing slab strength.

In adopting values for tension to be used in the corner-load formulas, it is desirable to use the highest reasonable value, as there is no objection to a small percentage of pavement failure. The values given at the close of this portion of the discussion have been tentatively adopted, as their use in conjunction with the distribution factors and wheel loads recommended seem to result in pavement thicknesses which have proved to be adequate under tests of modern traffic. These values are somewhat higher than many authorities advise.

The discussion of design tension values is based on the modulus of rupture of materials. The term "modulus of rupture" is defined as follows by Hool and Johnson:

"The transverse or beam strength of granular brittle materials like mortars and concretes is best expressed by the modulus of rupture. The modulus of rupture is the apparent stress in the extreme fiber of a transverse test specimen under the load which produces rupture. For specimens of rectangular section of breath b and depth d, loaded centrally on a span L, the breaking load being W, the modulus of rupture is computed by the formula

$$\text{Modulus of rupture} = \frac{3WL}{2bd^2}.$$

"The extreme fiber stress thus computed is not the actual fiber stress, because the formula involves the inaccurate assumption that the material deforms elastically for all stresses up to rupture. The comparative relations between results are not affected by this inaccuracy of the formula, however, when the tests compared are made upon specimens of similar material, because the computed values of the modulus of rupture are very nearly proportional to the actual stresses.

"Since the extreme fiber stresses on the tension side and on the compression side of a beam of homogeneous material are equal, the tensile strength of mortar or concrete is only a small fraction of the compressive strength, the transverse strength of mortar or concrete is almost wholly dependent upon the tensile strength. The modulus of rupture found in transverse tests will invariably be considerably in excess of the tensile strength, however, because the computed stress in the extreme fiber considerably exceeds the actual stress."

Modulus of Rupture of Plain Concrete.—For a number of years it has been quite well established that loads producing computed stresses exceeding 50 to 60% of the modulus of rupture of plain concrete result in permanent deflections. Recent experiments by the Illinois Highway Department on the effect of repetition of load on rupture have shown that loads producing computed stresses of 37% of the modulus of rupture can be repeated 1,000,000 times without causing failure; loads producing stresses of 53% of the modulus of rupture were repeated from 40,000 to 200,000 times before rupture occurred; loads producing stresses of 70% of the modulus of rupture broke the test piece after repetitions of from a few hundred to a few thousand applications To prevent rupture due to unlimited repetition of load, it is therefore desirable to adopt a design stress of about 45% of the modulus of rupture.

The modulus of rupture of concrete pavements or bases has considerable range in value, depending on the mix, age of concrete, and perfection of construction operations. It would be impracticable and uneconomical to attempt to use a design tension value which would prevent occasional failure of fresh concrete under repeated heavy loading.

New pavements are generally opened to traffic in 14 to 28 days after construction. It is necessary to provide enough strength at this time to prevent corner breaks due to the loading produced by normal traffic. The pavement must be able to handle immediately a few applications of the maximum permissible wheel load, but it is not rational to make the pavement strong enough at this stage to stand repeated rapid applications of the maximum legal load, nor is it likely that this condition will occur except under very unusual conditions.

The pavement must be strong enough when first opened to permit continual application of the loads produced by the usual vehicle, but these loads are far below the legal maximum. The maximum wheel pressure permitted by law is produced only by a very small percentage of the vehicles operating on the usual road. These vehicles in passing over the pavement apply their load only at critical corner points to a very small percentage of the slab corners. It is not likely that a single slab corner has the maximum legal load applied to the extreme corner with greater frequency than, say, one one-hundredth of 1% of the number of vehicles using the road The concrete rapidly gains strength with age, so that, when a design tension value is adopted high enough to prevent rupture at the 28-day age for the maximum load without considering the effect of repetition, the probabilities are all in favor that it will gain strength fast enough to take care of the maximum loads as they occur.

A value of about 65% modulus of rupture is, therefore, recommended for 28-day concrete in conjunction with the maximum legal wheel load. This means that the stress produced by the ordinary light vehicles is less than 50% of the modulus of rupture at 28-day age and can be repeated indefinitely even at that stage. Within 6 months or a year the concrete will probably gain sufficient additional strength to stand indefinite repetitions of the maximum legal load. If some corners break during the first few months, they can be repaired under the item of minor repair. For exceptional roads carrying a large volume of heavy trucking, it is, perhaps, desirable to reduce the design tension values somewhat, or to insist on longer ageing before the road is opened to traffic.

The following tabulation shows the approximate ratio of the design stress recommended for 1:1½:3 concrete to the modulus of rupture at 28 days, 6 months, and a year.

Recommended design tension value	Approximate ratio of recommended stress to modulus of rupture		
	28 days	6 months	1 year
400 lb. per square inch....	65%	50%	45%

The ratio of modulus of rupture to compressive strength (28-day) determined for ordinary road concrete mixes on 6 by 12″ cylinders is approximately 23% (see Table 75).

The tabulation below gives the recommended values for S tension to be used in the corner-load formula for plain concrete pavement and base design:

Adopted Values (S) Design Tensile Strength Value for Plain Concrete Pavements and Pavement Bases

1:1½:3 mix..........	400
1:2:4 mix..........	360
1:2½:5 mix..........	320
1:3:6 mix[1]..........	280

[1] Recent tests (1921) by the Bureau of Public Roads indicate that the strength of 1:3:6 concrete tested for beam action is about 80% of the strength of 1:1½:3 mix, while the ratio of direct compressive strength of these mixes is 1:2. The adopted values use a ratio of 70% for beam action.

Tables, 75, and 75A give reliable test values of the modulus of rupture and serve as the supporting data for the recommended values.

Table 76 compares direct tension with modulus of rupture.

TABLE 75.—EFFECT OF SIZE AND GRADING OF AGGREGATE AND AGE OF CONCRETE

(*Bulletin* 11, *Lewis Institute Chicago, Prof. Duff A. Abrams*)

Aggregates: sand from Janesville, Wis., and pebbles from Elgin, Ill. Aggregates of *different size* were obtained by separating sand and pebbles into various sizes and recombining as shown by sieve analyses in Table II and Fig. 1. *Different gradings* of aggregates were produced by mixing sand (0 to No. 4) and pebbles (No. 4 to 1½") in different proportions.

Mix, 1:4 by volume. Relative consistency, 1.10. Specimens tested damp. Each value is the average of five tests made on different days.

Ref. No.	Aggregate: Size	Aggregate: Fineness modulus	Water-ratio of concrete	Modulus of rupture of beams, pounds per square inch: 7 d.	28 d.	3 m.	1 y.	Compressive strength 6 by 12" cylinder, pounds per square inch: 7 d.	28 d.	3 m.	1 y.	Modulus of rupture, per cent of compression: 7 d.	28 d.	3 m.	1 y.
							Effect of Size of Aggregate								
36	0–16	1.95	1.29	95	160	255	340	270	620	1,190	1,600	35.2	25.8	21.5	21.2
37	0–8	2.17	1.25	95	195	320	370	360	850	1,470	1,860	26.4	23.0	21.8	19.9
38	0–4	2.45	1.20	125	250	370	425	430	1,010	1,620	2,100	29.4	24.8	22.8	20.2
39	0–⅜	4.00	0.98	290	455	595	640	1,040	2,110	2,930	4,190	27.9	21.6	20.3	14.3
40	0–¾	5.00	0.87	365	560	730	775	1,290	2,650	3,650	4,890	28.3	21.2	20.0	15.9
41	**0–1½**	**5.65**	**0.82**	**420**	**550**[a]	**810**	**880**	**1,410**	**2,580**[a]	**3,590**	**5,000**	**29.8**	**21.3**	**22.6**	**17.6**
			Aver.	230	360	510	570	800	1,640	2,410	3,320	29.5	22.9	21.5	18.2
							Effect of Grading of Aggregate								
29	0–1½	3.00	1.11	165	255	410	450	620	1,290	1,640	2,330	26.6	19.8	25.0	19.3
30	0–1½	4.00	0.98	230	390	505	570	950	2,000	2,550	3,230	24.2	19.5	19.8	17.7
31	0–1½	4.50	0.93	285	485	610	645	1,090	2,190	2,750	3,830	26.2	22.2	22.2	16.9
32	0–1½	5.00	0.87	325	505	660	710	1,160	2,410	3,580	4,510	28.0	21.0	18.4	15.8
33	0–1½	5.25	0.85	365	555	735	820	1,320	2,940	3,810	5,340	27.7	18.9	19.3	15.4
41	**0–1½**	**5.65**	**0.82**	**420**	**550**[a]	**810**	**880**	**1,410**	**2,580**[a]	**3,590**	**5,000**	**29.8**	**21.3**	**22.6**	**17.6**
34	0–1½	6.00	0.78	405	600	735	825	1,300	2,250	3,310	4,400	31.2	26.7	22.2	18.8
35	0–1½	6.25	0.77	385	590	730	865	1,140	1,990	2,840	4,080	33.8	29.6	25.7	21.2
			Aver.	320	490	650	720	1,120	2,210	3,010	4,090	28.4	22.4	21.9	17.8

NOTE.—Ordinary concrete pavement mixture shown in black-faced type (Reference 41).

[a] Average of 25 beam tests and 115 cylinder tests.

TABLE 75*A*.—TRANSVERSE STRENGTH OF MORTARS AND CONCRETES (*Tests of William B. Fuller*)

Proportions by weight, cement, sand, stone	Proportions by volume, cement, sand, stone	28-day age. Modulus of rupture, (pounds per square inch)		
		Maximum	Minimum	Average of 6
1:1: 2	1:1.17: 2.06	798	646	710
1:1: 3	1:1.17: 3.09	732	573	656
1:2: 4	1:2.34: 4.12	480	399	439
1:2: 5	1:2.34: 5.17	413	349	380
1:3: 5	1:3.51: 5.17	308	262	285
1:3: 6	1:3.51: 6.21	246	213	226
1:4: 8	1:4.68: 8.25	158	156	157
1:6:10	1:7.02:10.34	91	87	89

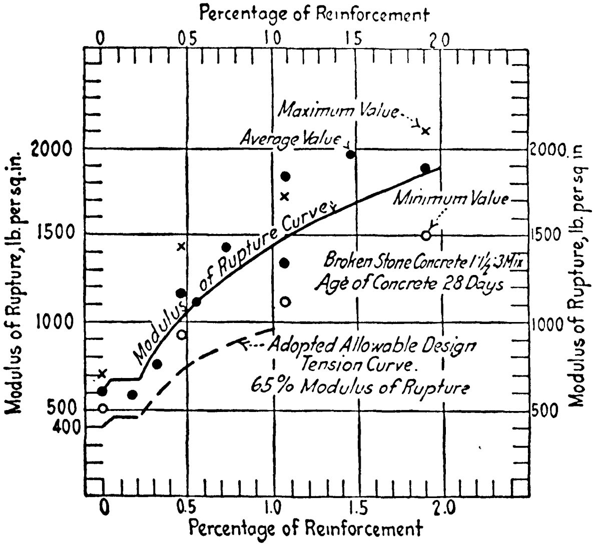
Table 76 compares the value of direct tensile strength tests with modulus of rupture.

TABLE 76

Mix	Ultimate tensile strength, pounds per square inch	Modulus of rupture, pounds per square inch
1:1½:3	300–450	500–700
1:2: 4	250–350	400–600
1:2½:5	200–300	350–450
1:3: 6	150–250	250–400

Modulus of Rupture of Reinforced Concrete.—The proper use of steel in road design is discussed on page 465. It is sufficient to note at this point that light mesh weighing from 0.25 to 0.65 lb. per square foot has been found useful in raising the average strength of the concrete throughout the slab, and that corner-bar reinforcement of sufficient area to raise the modulus of rupture materially at such points is common practice.

Experimental data on the effect of small percentages of steel on modulus of rupture are very meager. Experiments by the Bureau of Mines, Lewis Institute (Professor Abrams), and at Cornell give some basis for assumptions in this matter, but the experiments do not cover a very wide range of size and spacing of steel bars or wire mesh and are not conclusive as yet (see Tables 77*A* and 77*B*). The results indicate that light bar reinforcement having an effective area of less than two-tenths of 1% has practically no effect in increasing the modulus of rupture of the beam. Experience, however, shows that light mesh weighing about 0.25 to 0.4 lb. per square foot and having an effective area of about five-hundreths of 1% apparently has considerable effect in reducing corner cracks. This is probably due to its action as a fibrous tie in equalizing construction imperfections. The use of even the light mesh seems to warrant a slight increase in the value of *S* for infrequent internal crack corners held in close contact by longitudinal side-bar reinforcement. The use of corner-bar reinforcement having an effective area of 0.2% or more has a decided value as determined by both experiment and practice. The effect of different percentages



Percentage Reinforcement	Adopted Allowable Tension Value (S) 1:1½:3 Concrete
0.0	400 lbs. per sq. in.
0.1	460 " " " ".
0.2	470 " " " "
0.3	580 " " " "
0.4	670 " " " "
0.5	760 " " " "

FIG. 135.—Effect of steel reinforcement on modulus of rupture of concrete.

of reinforcement on modulus of rupture and the tentative recommended values for S tension are shown in Fig. 135. These values are based on Table 77.

TABLE 77A.—EFFECT OF NUMBER OF STEEL REINFORCING BARS
(*Bulletin* 11, *Lewis Institute Chicago, Prof. Duff A. Abrams* 1922)

Beams: Depth, 7″; width, 10″; span, 36″.
Longitudinal reinforcing bars were spaced equally across the width and 1″ from the bottom of the beam. The total depth of beams varied from 6.9 to 7.1″.
Mix, 1:4 by volume. Aggregate: sand and pebbles graded 0 to 1½″.
Relative consistency of concrete, 1.10; water ratio, 0.82.
Age at test, 28 days.
Specimens tested damp.
Each value is the average of five tests made on different days.
Compressive strength by 6 by 12″ concrete cylinders (average of 115 tests) was 2580 lb. per square inch.

Reference number	Number of ⅜″ round steel bars	Percentage of steel	Total load on beam, pounds	Computed stresses, pounds per square inch					Type of beam failure
				Tension in steel	Compression in top fiber	Bond	Shearing unit stress	Modulus of rupture	
7, 8	0	None	7,870[a]	. . . . .	. . . .	. . .	70	550[a]	Tension
59	1	0.18	7,850	73,000[b]	1,470[b]	570	70	560[b]	Tension
60	2	0.36	10,390	49,400[b]	1,460[b]	380	90	730[b]	Tension
61	3	0.54	15,010	49,100[b]	1,850[b]	380	140	1,085[b]	Tension
62	4	0.72	19,640	48,900[b]	2,190[b]	380	180	1,435[b]	Diagonal shear
63	6	1.09	25,820	43,800[b]	2,160[b]	340	240	1,885[b]	Diagonal shear
64	8	1.46	28,060	36,800[b]	2,530[b]	290	270	1,990[b]	Diagonal shear

[a] Average of 25 beam tests.
[b] Tension in concrete not considered in computing steel stress. Steel not considered as taking stress in computing modulus of rupture.

TABLE 77*B*.—RECORD OF TESTS AT CORNELL UNIVERSITY ON REINFORCED-CONCRETE BEAMS 5 × 7″, SPAN 4.0′ CENTRAL LOAD (AGE 28 DAYS)

Proportions by volume	Per cent of reinforcement	Maximum	Minimum	Average
Gravel concrete				
1.5	0.47	887	722	844 (4 tests)
1:5	1.07	1262	826	1032 (4 tests)
1:5	1.89	1300	1020	1175 (4 tests)
Crushed-stone concrete				
1:1½:3½	0.47	1430	916	1159 (6 tests)
1:1½:3½	1.07	1720	1123	1344 (6 tests)
1:1½:3½	1.89	2100	1500	1880 (6 tests)

Even with considerable more experimental data, the effect of steel in actual design would be open to uncertainty for the following reasons: Road slabs are so thin that even small variations in location of the steel upsets its theoretical effect as regards tension. Even with the most intelligent labor and inspection, considerable variation occurs, so that the net practical value of steel as a tension. reinforcement is open to considerable uncertainty. For the thicker slabs, 7 to 8″, reasonably close assumptions can be made. These depths give bars some chance actually to work. For slabs less than 7″ deep steel is of little value in raising tension. In these slabs its value is confined to equalization and contact uses, as is later discussed. For the purposes of this discussion the relation between depth of slab and theoretical tensile effectiveness of steel is assumed as follows:

TABLE 78

Depth of slab, inches	Assumed effectiveness *S*	
	Mesh reinforcement, per cent theoretical[a]	Corner bars, per cent theoretical
7½ or more	100	100
7	100	90
6½	80	60
6	50	0

[a] Light-mesh reinforcement can be safely placed nearer the surface of the concrete than ½ or ⅝″ bar reinforcement.

The actual value of steel is really more a matter of cut and try than it is of theoretical design, but these statistics serve as an basis for judgment.

Wheel-load Factor W.—The wheel loads were discussed on page 373. The reader is referred back to that data, which recommended the design loads shown in Table 79 for Class I and II traffic. These recommended design wheel loads include an allowance for impact and apply to districts in which the gross vehicle load is limited to 28,000 lb.

TABLE 79.—TABLE OF RECOMMENDED DESIGN WHEEL LOADS FOR DISTRICTS HAVING A SINGLE-VEHICLE, 28,000-LB. LEGAL MAXIMUM LIMIT (CLASS I AND II TRAFFIC)

Type of pavement	Design wheel[a] load W, pounds
Asphaltic concrete on cement-concrete base	12,000
Asphalt block on cement-concrete base	14,000
Plain or reinforced cement concrete	14,000
Brick (cement-grout) concrete base	14,000
Monolithic brick	14,000
Brick (bituminous-filled) bit-sand cushion	14,000
Brick (bituminous-filled) cement-sand cushion	15,000
Stone block	16,000

[a] The differences are due to variation in impact allowance for different surfaces. See page 373.

Figures 138 to 143, which show graphically the theoretical depths obtained from the use of the corner-load formula, consider a range of design wheel load from 6000 to 20,000 lbs. As a rule, 9000 lb. is the least design load applicable to actual traffic conditions and 16,000 lb. the maximum. There are unusual cases, however, where 20,000 lb. load does occur today (1922, Massachusetts Traffic Report). Such loads violate legal statutes and their occurrence shows the necessity for police regulation unless adequate depth provision is made for their occurrence.

Location and Distribution of Load at Joints J.—Both the location and the distribution of loads affect the value of J (effective load applied at extreme corner of slab).

Critical Location of Load.—There are two general classes of slab corners, exterior and interior (Fig. 136). Wheel loads can be applied very close to the extreme corner along the outer edges of pavements with earth shoulders. It is impossible to apply the full wheel load at interior corners closer than about 6″ from the extreme point, provided the adjacent slabs are at the same elevation. This means that it is not possible for the load moment to be as great at interior corners as along the outer edge of pavements having earth shoulders. This condition in conjunction with other factors later discussed indicates in a general way that monolithic rural highway pavements constructed without raised curb edges should be designed stronger along the outer edge than along central joints. This conclusion is borne out by observed failures, fewer of which occur along central joints than along the outer edge. Recent designs recognize this fact by specially strong outer-edge design (see p. 455). City-street pavements with raised curbs have only one class of corner, namely, interior classification.

The exact effect of this difference in location of load on values for J is not known, but a rough approximation can be made. Corner cracks rarely start at more than 30 to 36″ away from the corner point, and generally at less distance. If 36″ is assumed as extreme distance, a difference of 6″ in moment arm would reduce the load moment about 15% for interior corners, which is a conservative allowance. The finally adopted values for J may well be about 15 to 20% less for interior than for exterior corners. Distribution of load at joints is, however, the main factor in determining reasonable values for J.

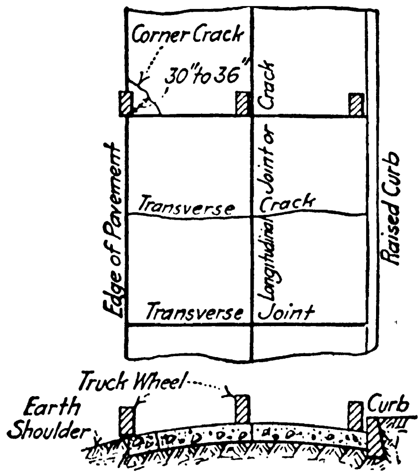


FIG. 136.—Location of critical wheel loads at internal and external corners.

Distribution of Wheel Loads at Joints.—The distribution of load between adjacent slabs at joints and cracks is a variable and indefinite factor. In monolithic pavements it depends largely on the degree of contact, which is much better during the day than at night, and better in the summer than in the winter. The use of steel-tie members in concrete increases the contact, at least temporarily (see p. 471). For pavements with separate concrete bases and top surfacing courses, like asphaltic concrete, semi-monolithic brick, etc., it is certain that the distribution of load over base cracks is much better than for the monolithic-slab type. Most of the experimental data deal with distribution at joints in monolithic concrete and brick These types will be given first consideration and the values adopted can then be modified for the other types by data which compare this factor in an indirect manner.

Monolithic-slab Joint Distribution.—Table 80, taken from an article by H. F. Clemmer, of the Illinois Department of Highways, and C. A. Hogentogler, U. S. Bureau of Public Roads, contains

TABLE 80.—OBSERVED AND COMPUTED DEFLECTIONS OF SLAB CORNERS ADJACENT TO JOINTS

Section			October night observed deflection				October day observed deflection			
No.	Thickness, inches	Kind	Load on near side	Load on far side	Per cent load carried across joint	Total movement of corners	Load on near side	Load on far side	Per cent load carried across joint	Total movement of corners
40	9	Conc.	0.081	0.024	13.0	.....	0.040	0.019	35.0	
40	9	Conc.	0.066	0.012	26.0	0.111	0.042	0.021	30.0	0.042
40	9	Conc.	0.041	0.040	49.0	.....	0.035	0.035	50.0	
39	8	Mono.	0.040	0.039	49.0	0.002	0.035	0.035	50.0	0.0
42	8	Conc.	0.078	0.037	43.0	.....	0.030	0.030	50.0	
43	7	Conc.	0.050	0.030	26.0	0.061	0.030	0.030	50.0	0.0
63B	7	Conc.	0.075	0.026	15.0	.....	0.055	0.026	18.0	
63A	7	Conc.	0.070	0.015	29.0	0.104	0.047	0.014	38.0	0.062
44	7	Conc.	0.094	0.004	11.0	.....	0.060	0.011	18.0	
45	6	Conc.	0.106	0.009	5.0	0.187	0.076	0.016	15.0	0.109
38	7	Mono.	0.045	0.045	50.0	.....	0.037	0.037	50.0	
37	6	Mono.	0.045	0.045	50.0	0.00	0.037	0.037	50.0	0.00
48	6	Conc.	0.085	0.035	47.0	.....	0.048	0.043	41.0	
47	6	Conc.	0.092	0.060	20.0	0.082	0.044	0.038	46.0	0.011
51	6	Conc.	0.114	0.095	20.0	.....	0.076	0.059	28.0	
50	6	Conc.	0.105	0.063	33.0	0.061	0.079	0.050	44.0	0.046
52	6	Conc.	0.094	0.053	15.0	.....	0.086	0.050	49.0	
51	6	Conc.	0.101	0.027	46.0	0.115	0.073	0.008	46.0	0.101

TABLE 90.—(*Continued*)

Section			Including per cent distribution day and night	December total movement day and night	Computed equal deflections for both slabs at joint					
					Rear wheel			Front wheel		
No.	Thickness, inches	Kind			Night	Day	Difference	Night	Day	Difference
40	9	Conc.	22.0							
40	9	Conc.	4.0	0.069	0.046	0.031	0.015	0.016	0.008	0.008
40	9	Conc.	1.0							
39	8	Mono.	1.0	0.002	0.040	0.035	0.005	0.009	0.006	0.003
42	8	Conc.	7.0							
43	7	Conc.	24.0	0.061	0.047	0.030	0.017	0.011	0.007	0.004
63B	7	Conc.	3.0							
63A	7	Conc.	9.0	0.042	0.046	0.038	0.008	0.016	0.008	0.008
44	7	Conc.	7.0							
45	6	Conc.	10.0	0.079	0.057	0.040	0.017	0.016	0.007	0.009
38	7	Mono.	0.0							
37	6	Mono.	0.0	0.00	0.045	0.037	0.008	0.012	0.008	0.004
48	6	Conc.	6.0							
47	6	Conc.	26.0	0.071	0.070	0.044	0.026	0.025	0.011	0.014
51	6	Conc.	8.0							
50	6	Conc.	11.0	0.015	0.098	0.066	0.432	0.028	0.018	0.010
52	6	Conc.	35.0							
51	6	Conc.	0.0	0.012	0.078	0.057	0.031	0.024	0.015	0.009

NOTE.—Equal deflections of both corners at joints indicates perfect contact, or 50 per cent distribution.

probably as reliable data as can be obtained at present (1922). This table shows a wide range in values.

It is possible that under favorable conditions the load may be equally distributed between adjacent slabs, but this is not likely to be the case for many cases at cracks in the plain concrete-slab type or at expansion joints filled with bitumen. For the most adverse conditions of contact, a single corner may carry the entire wheel load. At expansion joints it is not unlikely that this condition occurs at intervals, but as the factor of safety of the formula is to be provided by the soil-contact factor, it is not advisable to assume the highest possible value for J. Values are derived as follows for plain concrete pavements:

As defined on page 395, $J = 3 \times$ the effective part of the wheel load actually applied at the extreme corner. The effective part is made up of the part of wheel load W carried by a single corner modified for location of this load. The extreme possible range for exterior corners ranges from $1.0W$ to $0.5W$ applied at the extreme point. This would give a possible range of value for J at exterior corners as shown in the following tabulation.

Wheel load carried by single corner	Value of J (exterior corners)
$0.5W$	1.5
$0.6W$	1.8
$0.7W$	2.1
$0.8W$	2.4
$0.9W$	2.7
$1.0W$	3.0

The value of 2.6 is considered safe and reasonable for exterior-corner condition in plain concrete-slab design. Interior-corner value is reduced 15%, giving a value of 2.2 recommended.

In pavements containing imbedded steel the distribution at expansion joints is the same as for the plain concrete type. Where joints or cracks not subject to expansion movement are held in contact by tie bars or dowels, the distribution is materially increased. Some designers advocate values for J based on a 50% distribution, but this seems rather optimistic. A value for exterior corners is recommended based on $0.7W$, which equals 2.1 reduced to 1.7 for interior corners.

The adoption of values for J as applied to two-course pavements, such as asphaltic concrete, block pavements, etc., depends on indirect comparison of the action of these pavements with the monolithic type under traffic and experimental beam tests. These values for two-course pavements are still in the speculative stage, but if they are adjusted to produce rational results in conjunction with the values for W and S which are fairly certain, there is some justification for the use of the corner-load formula in connection with the design of two-course pavements.

Any top course adds something to the resistance of the concrete base by its effect on wheel impact, increase in the reliability of load distribution over base cracks, reduction in probability of load being applied at the extreme corner of the base slab, and reduction in probability of surface-water seepage through cracks to the underlying subgrade. Of these considerations, J is affected only by distribution and location of load.

The recommended values are based on the consensus of opinion of engineers in western New York and on experimental results of the Bates Road Tests and the following quotation from an article by C. A. Hogentogler, U.S. Bureau of Public Roads, November, 1921, summarizing recent Arlington experimental tests:

"The monolithic brick-concrete slabs in most cases showed less resistance than the 1:1½:3 concrete slabs of the same depth. Failure of the former seemed to result from the shearing of the brick top from the concrete base before full resistance of the monolithic was developed. This allowed the specimens to develop at most only the sum of the resistance of the two parts. The monolithic sections tested as beams under static loads failed in the same way, but the resistance of the beams seemed to be slightly in excess of the sum of the top and bottom strengths, while the slab resistances compared favorably with the sum of the resistances of the two parts. This would indicate that higher shearing stresses are developed by impact than by equivalent static loads.

"With possibly one exception (slab 106), the grouted-brick tops with sand-cement cushions or concrete bases showed less resistance than the monolithic sections.

"Grout-filled brick tops with sand and screening cushions on concrete bases showed slightly greater resistance than would be expected from the bases alone.

"Grout-brick surfaces compared favorably with 1:1½:3 slabs of equal thickness, while grouted-brick beams showed resistances in excess of those offered by equal thicknesses of 1:1½:3 concrete.

"As would be expected, the beam strengths of the bases were not much increased by bitumen-filled brick to brick tops."

TABLE 81.—RECOMMENDED VALUES J TWO-COURSE PAVEMENTS

Type of pavement	Value of J, interior-corner classification
Asphaltic-concrete pavements	1.2–1.3
Block pavements (bituminous joints)	1.2–1.3
Block pavements (cement-grout joints)	0.8–0.9

RECOMMENDED VALUES J MONOLITHIC PAVEMENTS

Type of pavement	Value J, exterior corners	Value J, interior corners
Plain concrete	2.6	2.2
Monolithic brick	2.6	2.2
Reinforced concrete		
At expansion joints	2.6	2.2
At cracks or contraction joints with bar ties	2.1	1.7

Soil Support (Percentage Modification Factor P).—As this is the most indefinite element of the problem, it seems desirable to adopt values which are on the safe side and in this way provide the necessary factor of safety for the formula. The depths directly computed by the formula $d = \sqrt{\frac{JW}{S}}$ assume complete loss of soil support. If soil support is given some consideration in design, it is recognized by an arbitrary percentage modification of depths and the percentage used must agree with the observed action of pavements under traffic tests on different subsoils. For practical application to design the formula may be written as

$$d = P\sqrt{\frac{JW}{S}},$$ where P represents the soil-support factor.

Engineers differ considerably in regard to the allowable reduction in depth due to partial soil support; they are, however, practically unanimous in agreement that some allowance should be made for the factor, particularly where a rigid pavement is laid on top of an old, firm macadam road in reconstruction programs. Where really unstable subgrades, such as muck or quicksand, are encountered, it is necessary to stabilize the subgrade by means of gravel or cinder subbase well underdrained before the rigid pavement can be laid with any degree of success. The value assigned to P has an important effect on the practical value of the formula.

If the complete loss of soil support were a common continuous occurrence on the ordinary subgrade soils, such as loams and the stiffer clays, there would be a larger percentage of failure on some of the old thin rigid pavements than has actually been the case. This indicates that these soils usually give some support. On the other hand, there have been enough observed failures to indicate that it is not safe to assume much support from these soils under adverse conditions. The following short quotation from *Bulletin* 18, "Bates Experimental Road," gives experimental data in connection with the loss of soil support due to temperature warping.

"The curling of the edges of a concrete slab 18′ wide may be sufficient to render the most perfect of subgrades ineffective under the edges of the slab. This can be readily determined without the aid of accurate measuring devices. To visualize this condition the line drawings shown on page 411 which are based on photographs were made.

Best available data indicate that monolithic pavements having slab widths of 10′ or less and the usual two-course pavements having widths of 20′ or less generally receive some benefit from support at edges and corners. Experience indicates that if we adopt values for P of 1.1 for wide monolithic slabs and values of 0.8 to 1.0 for narrow slabs and two-course pavements the results are safe. The following values for P depending on the character of the subgrade seem to be about as good a guess as can be hazarded at present (1923).

TABLE 82.—RECOMMENDED VALUES (P) (SOIL-SUPPORT FACTOR)

Subgrade conditions	Type of pavement	Adopted formula
Ordinary loams and clays.	Wide monolithic slabs	$d = 1.1\sqrt{\frac{JW}{S}}$
	Narrow slabs or two-course pavements	$d = 1.0\sqrt{\frac{JW}{S}}$
Coarse gravels or macadams.	Wide monolithic	$d = 1.0\sqrt{\frac{JW}{S}}$
	Narrow slabs or two-course pavements	$d = 0.9\sqrt{\frac{JW}{S}}$
Quicksand and muck	Wide monolithic	Subbase $+ d = 1.1\sqrt{\frac{JW}{S}}$
	Narrow monolithic or two-course pavements	Subbase $+ d = 1.0\sqrt{\frac{JW}{S}}$

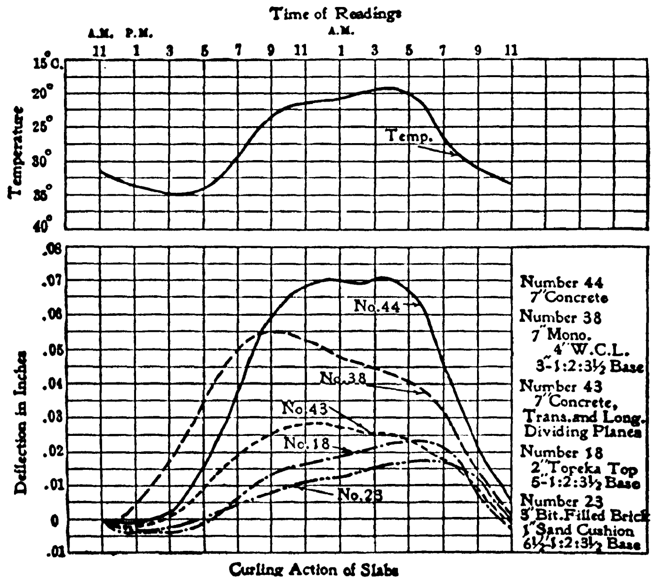


FIG. 137.—Bates road experiments on rigid slab warping due to temperature.

Summary of Discussion of Factors.—The preceding discussion outlines the wide range of possible values for each of the factors in the formula $d = P\sqrt{\frac{JW}{S}}$. There is enough variation in these values to give some theoretical justification for practically any depth in common use. The range in values shows that proper design depends very largely on the judgment of the designer. The student may well bear in mind, however, that many engineering formulas, such as sewer run-off, etc., which have been found very useful in practice, are susceptible to the same variation in final results and that the values of such formulas have been gradually increased by modifying experimental factor values to agree with actual service tests. A formula of this nature is very useful in coordinating the experience of a large number of different men working on the same general problem, as it affords a convenient means of expressing their experience in mathematical form.

The practical use of the corner-load formula is illustrated in the following pages for each common type by showing the range in possible values, giving a recommended set of values for normal conditions and computing the depths required for different wheel-load limits. These results are compared with ordinary practice. It is not likely that the recommended depths can be safely changed more than 5 to 10% for normal conditions. For special traffic conditions changes can be made in the factor values, but it is not desirable to use a combination of all high or all low values.

Plain Concrete Pavements.—Pages 449 to 474 give a discussion of joint details, richness of mix, etc. Design is based, as a rule, on a system of longitudinal and transverse joints which divide the pavement into slabs of from 8 to 12′ width and from 30 to 40′ length, which usually control the natural tendency of the pavement to crack under temperature and frost action. That is, some intermediate cracking will occur, but the intersection of joints and the infrequent internal cracking are at approximately 90° angles. Richness of mix varies from 1:2:4 on unimportant roads to 1:1½:3 on main heavy-traffic roads. Factor values and resultant depths are indicated as follows for the formula $d = P\sqrt{\frac{JW}{S}}$:

RECOMMENDED FACTOR VALUES FOR ROADS OF CLASSES I AND II IN DISTRICTS PERMITTING A 28,000-LB. GROSS VEHICLE ROAD

Design wheel load, including impact W, pounds..........	14,000
Load distribution J, exterior corners....................	2.6
Load distribution J, interior corners....................	2.2
Design tensile stress S, 1:1½:3 mix, pounds per square inch	400
Design tensile stress S, 1:2:4 mix, pounds per square inch	360
Soil-support factor P, ordinary soils....................	1.0
Soil-support factor P, gravel and macadam..............	0.9

These values result in the following recommended theoretical depths for 1:1½:3 mix (28,000-lb. vehicle load).

Ordinary foundation soils:

$$\text{Outer edge of pavement } d = 1.0\sqrt{\frac{2.6(14,000)}{400}} = 9.5''$$

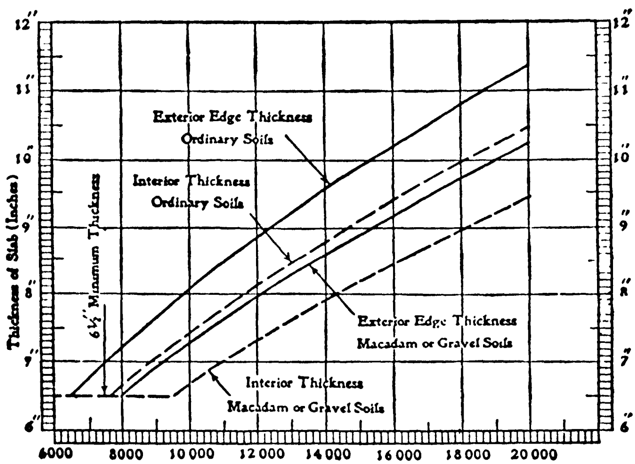
$$\text{Interior areas}^1 \text{ of pavement } d = 1.0\sqrt{\frac{2.2(14{,}000)}{4}} = 8.8''$$

Gravel or macadam foundation:

$$\text{Outer edge of pavement } d = 0.9\sqrt{\frac{2.6(14{,}000)}{400}} = 8.5''$$

$$\text{Interior areas}^1 \text{ of pavement } d = 0.9\sqrt{\frac{2.2(14{,}000)}{4}} = 7.9''$$

Variations in depth due to changes in the design wheel load are shown in Fig. 138 below.



Design Wheel Load in lbs. (Includes Impact Allowance)
3½ ton truck (16,000 lbs. gross load) Design Wheel load 9500 lbs.
5 ton truck (22,000 lbs. gross load) Design Wheel load 11,500 lbs.
7 ton truck (28,000 lbs. gross load) Design Wheel load 14,000 lbs.

FIG. 138.—Theoretical thickness plain concrete slabs 8 to 12 ft. wide 1:1½:3 mix (crushing strength 28-day age 3000 to 4000 lb. per square inch).

Ordinary practice in plain concrete road design uses average depths of from 5½ to 11″ (see Table 84, p. 417).

Where pavements are constructed over new trenches (culverts or sewers) which will surely settle, the slab depths at such localized places are designed on the principle of bridge slab depths with extra bottom reinforcement (see p. 417).

While a considerable mileage of plain concrete pavements has been constructed and is serving moderately well, the tendency is to use some steel, as considered in the next section.

Reinforced-concrete Pavements.—These pavements are designed on the same general arrangement of joints as the plain concrete

[1] Interior-area classification applies to total width of curbed streets.

except that the length of the slab is usually somewhat greater (40′ being a common length).[1] The following theoretical depths consider steel 100% effective for tie purposes but of little effect in increasing tensile strength for slabs less than 6½″ thick. The depth of the pavement is computed on the basis of corners at intermediate cracks between expansion joints where good contact is fairly certain. Corners at expansion joints are given the same thickness as close contact corners and the necessary additional bar reinforcement is provided to take care of the extra stress due to poorer joint distribution.

RECOMMENDED FACTOR VALUES. ROADS OF CLASSES I AND II IN DISTRICTS PERMITTING A 28,000-LB. GROSS VEHICLE LOAD

Design wheel load including impact W, pounds	14,000
Load distribution J, exterior corners	2.1
Load distribution J, interior corners	1.7
Load distribution J, exterior expansion joints	2.6
Load distribution J, interior expansion joints	2.2
Design tensile strength for all parts of slab except at expansion joint corners, assuming 0.3- to 0.4-lb. mesh reinforcement per square foot and side tie bars:	
1:1½:3 mix, pounds	450
1:2:4 mix, pounds	420

If mesh is omitted and tie bars alone used, the tension value S is recommended at 400 lb. per square inch (1:1½:3 mix) and 360 lb. (1:2:4 mix), using J values for tight contact (see Fig. 140).

Soil-support factor P, ordinary soils	1.0
Soil-support factor P, gravel or macadam	0.9

This results in the following recommended depths and corner reinforcement for 1:1½:3 mix (28,000-lb. gross vehicle load).

Ordinary foundation soils:

$$\text{Outer edge of pavement } d = 1.0\sqrt{\frac{2.1(14{,}000)}{450}} = 8.1''$$

$$\text{Interior areas}^2 \text{ of pavement } d = 1.0\sqrt{\frac{1.7(14{,}000)}{450}} = 7.3''$$

Design tensile stress at expansion joint corners:

$$\text{Outside corner } S = \frac{JW}{d^2} = \frac{2.6(14{,}000)}{8.1^2} = 550 \text{ lb.}$$

$$\text{Inside corner} \qquad = \frac{2.2(14{,}000)}{7.3^2} = 580 \text{ lb.}$$

[1] On account of future surfacings of bituminous concrete 33′ is probably a better spacing to reduce localized movement.

[2] Interior-area classification applies to total width of curbed streets. Design tensile stress at expansion-joint corners.

Figure 135 (p. 401) indicates that a design stress of 580 lb. requires approximately 0.3 to 1% area of reinforcement. This applies for the sectional area of the slab back about 30″ from the extreme corner. Size and spacing of corner bars are based on these data.

Gravel or macadam foundation:

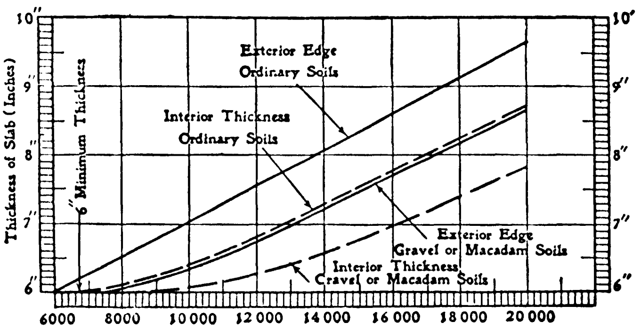
$$\text{Outer edge of pavement } d = 0.9\sqrt{\frac{2.1(14{,}000)}{450}} = 7.3''$$

$$\text{Interior areas[1] of pavement } d = 0.9\sqrt{\frac{1.7(14{,}000)}{450}} = 6.6''$$

$$\left.\begin{aligned} &\text{Outside corners } S = 0.8\frac{JW}{d^2} = 0.8\frac{2.6(14{,}000)}{7.3^2} = 550 \text{ lb.} \\ &\text{Interior corners} \qquad\quad = 0.8\frac{2.2(14{,}000)}{6.6^2} = 570 \text{ lb.} \end{aligned}\right\} \text{Expansion joints.}$$

Requires three-tenths of 1% reinforcement.

Concrete 1 : 1½ : 3 Mix Crushing Strength (28 days) 3000 to 4000 lbs.
Reinforcement—Mesh and Bar.
Mesh 0.3 to 0.4 lbs. per sq. ft.
Total Corner Steel 0.3 of one % Section area for 30″ from Corner
Side Tie bars (See page 451)
Central Longitudinal Joint.



Design Wheel Load in lbs. (Include Impact Allowance)

3½ ton truck (16,000 lbs. gross load) Design Wheel load 9,500 lbs.
5 ton truck (22,000 lbs. gross load) Design Wheel load 11,500 lbs.
7 ton truck (28,000 lbs. gross load) Design Wheel load 14,000 lbs.

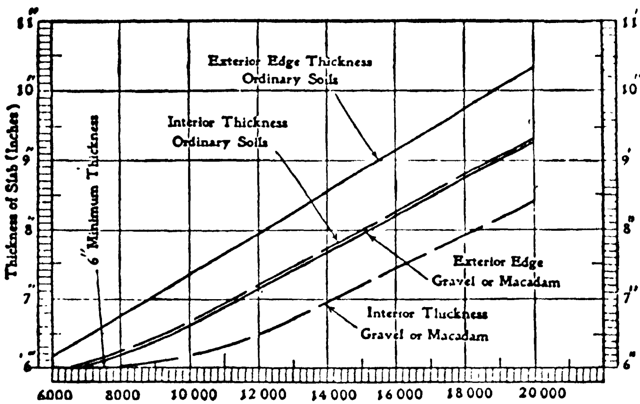
Note: The difference in the general shape of this curve from the plain Concrete curve is due to the decreasing effectiveness of steel for slabs less than 7″ thick

FIG. 139.—Theoretical thickness reinforced cement concrete. 1 : 1½ : 3 mix, mesh and bar reinforcement, transverse and longitudinal joints.

[1] Interior-area classification applies to total width of curbed streets. Design tensile stress at expansion-joint corners.

Theoretical variation in depth due to changes in the design wheel load are shown in Figs. 139 and 140 for 1:1½:3 mix with different kinds of reinforcement.

Concrete 1:1½:3 Mix Crushing Strength (28 days) 3000 to 4000 lbs.
Reinforcement Side Tie Bars (See page 453)
Corner Bar Reinforcement 0.3 of one % Section Area for 30″ back from Corner
Central Longitudinal Joint.



Design Wheel Load in lbs. (Includes Impact Allowance)

3½ ton truck (16,000 lbs. gross) Design Wheel load 9,500 lbs.
5 ton truck (22,000 lbs. gross) Design Wheel load 11,500 lbs.
7 ton truck (28,000 lbs. gross) Design Wheel load 14,000 lbs.

FIG. 140.—Theoretical thickness reinforced concrete 1:1½:3 mix, side tie bar and corner bar reinforcement, transverse and longitudinal joints.

Effect of Steel on Depth.—A comparison of Figs. 138, 139, and 140 indicates that the use of tie and corner bars reduces the required thickness of plain concrete slabs about 1″ and that the use of mesh, tie bars, and corner bars reduces the required depth of concrete about 1½″. This applies only for load conditions requiring depths of plain concrete of 8″ or more. Expressed in terms of money, tie and corner bars reduce the cost of concrete about 4 cts. a square foot (1922 cost conditions) and tie, corner bars, and mesh reduce the cost of the concrete about 6 cts. per square foot. The cost of the steel used as reinforcement should not exceed these limits. The cost of steel reinforcement is usually less than this amount, which indicates that the use of steel is economically desirable.

Ordinary practice in reinforced-concrete-pavement design uses average depths of from 5 to 8″. For relative depths of sides and edges and reinforcement details see page 455.

Depths over Trenches.—Where pavements are constructed over new trenches (culverts or sewers) which will surely settle, the slab depths at such localized places should be thickened and given additional bottom reinforcement to conform with bridge-floor slab practice. The following table indicates in a general way the maximum width of trench it is safe to span with standard road slabs of 1 : 1½ : 3 mix for different legal load limits based on formula (safe span) $= \frac{32S(d)^2}{W}$:

TABLE 83.—TRENCH WIDTHS REQUIRING ADDITIONAL STRENGTH

Maximum load	Approximate maximum allowable trench width safe to span without additional thickening (1 : 1½ : 3 mix), in feet Column headings are depths of standard pavement slabs			
	6″	7″	8″	9″
3½-ton truck (16,000 lb. gross)	4.0′	5.5′	7.5′	9.5′
5-ton truck (22,000 lb. gross)	3.5′	5.0′	6.5′	8.0′
7-ton truck (28,000 lb. gross)	3.0′	4.5′	5.5′	6.5′

NOTE.—These widths assume beam action, that is, joints must not be constructed directly over a trench.

TABLE 84.—TABLE OF RANGE IN DEPTH OF CONCRETE PAVEMENTS. CURRENT PRACTICE, 1919–1922, IN INCHES

State	Plain concrete	Reinforced concrete
Arizona	5½– 9	....
California	6	5 – 6
Connecticut	6 – 9	....
Illinois	....	6 – 9
Indiana	7 – 8	7 – 8
Maryland	6½– 8	....
Massachusetts	....	7½– 8
Minnesota	....	7½
New Jersey (1922)	8	8
New Jersey (1919)	6 –10½	....
New York	....	6 – 7
Pennsylvania	7 – 9	6 – 8
Rhode Island	6 – 8	6 – 8
Washington[a]	6 – 11[a]	6 –11[a]
West Virginia	6 – 9	....
Wisconsin	7 – 8	....

[a] State of Washington, 11″ depth for short distances where settlement may occur.

NOTE.—The preceding theoretical analysis indicated the following depths as reasonable for a 28,000-lb. gross vehicle load, ordinary foundation soils:

Plain concrete (1 : 1½ : 3 mix).................... 8¾–9¾″
Reinforced concrete (1 : 1½ : 3 mix).............. 7½–8½″

Trend in Design.—A comparison of Table 84 on current practice depths with the preceding discussion of recommended theoretical depths shows that, with the exception of a few of the states, the maximum thickness in common use is somewhat less than the theoretical depths developed for a 28,000-lb. gross vehicle limit.

These theoretical depths are based on the maximum statutory load adopted in a number of the thickly settled states, as it is believed that loading on main roads will tend to increase to the limit permitted by law. As previously discussed, the rigid type of pavement is rarely economical on moderate or light-traffic roads, so the theoretical depths have been developed to represent Class I traffic requirements. The states using depths as developed by these formulas are generally thickly settled with large cities and a heavy volume of commercial trucking on the main intercity routes. It is believed that a rigid pavement is not usually justified unless such conditions prevail, but a great many localities are building lighter concrete roads for lighter traffic. If the designer is willing to recognize that he is taking a chance and considers the concrete pavement as a temporary expedient with the idea of capping it in a short time with some standard renewable surface, it may be justifiable to reduce the depth to that required for two-course pavement bases. Under this line of reasoning the following minimum depths have some justification, on the basis that the proposed future surfacing will raise the strength of the finally completed pavement up to that required for a 28,000-lb. gross vehicle load.

Plain concrete:	
1:1½:3 mix	6½″
1:2:4 mix	7 ″
Reinforced concrete:	
1:1½:3 mix	6 ″
1:2:4 mix	6½″

It, however, seems undesirable to carry this idea too far, as it tends to discredit this type of pavement, which is very useful and economical if properly designed for roads carrying from 2000 to 6000 vehicles daily.

Bituminous-Concrete Surface on Cement-Concrete Base.—Cement-concrete bases for these pavements are usually constructed as a continuous slab without any special provision for expansion or contraction. These bases crack with more or less regularity transversely across the pavement at intervals of 30 to 60′, and more infrequently in a longitudinal line near the center of the pavement and roughly parallel with the edges; that is, the action of the elements combined with traffic produces a series of slabs of varying sizes and shapes. Cracks in the asphaltic-concrete surface often develop directly over the base crack, but do not always show through to the surface, which depends largely on the width of the base crack. Wide base cracks are not so frequent for the lean concretes as for the rich concretes because the coefficient of expansion is considerably less for a 1:3:6 than it is for a 1:2:4 mix (see

THE TREND OF CONCRETE PAVEMENT DESIGN BY YEARS FROM HIGHWAY RESEARCH NEWS, AUGUST, 1926

(Tabulation by U. S. Bureau of Public Roads. Based on Federal-Aid Prospects)

Pavement thickness (inches)			Number of projects submitted by the 48 states by years								
Edge	Center	Edge	1917	1918	1919	1920	1921	1922	1923	1924	1925
5	5	5	...	1	...	1	2	1	...	...	...
6	6	6	2	3	3	24	24	49	10	4	3
7	7	7	1	11	17	31	22	80	70	33	47
7½	7½	7½	2	4	17	31	21	30	9	1	...
8	8	8	4	17	68	90	78	85	61	51	44
9	9	9	...	1	13	9	9	8	8	...	...
10	10	10	...	...	5	5	5	2	2	...	1
12	12	12	...	...	...	...	...	...	2	...	1
5	6	5	...	...	1	11	3	1	...	...	...
5	7	5	4	10	7	4	...	3	...	...	...
6	7	6	...	1	9	17	8	39	25	...	...
6	8	6	4	13	92	60	56	71	16	25	9
7	8	7	1	11	65	55	23	24	18	...	...
7	9	7	...	...	1	2	...	2	4	...	...
8	10	8	...	...	...	...	...	1	1	...	...
7	5	7	...	...	...	...	...	1	...	...	...
7	6	7	...	...	...	...	...	1	23	38	43
7½	5½	7½	...	...	...	...	...	...	...	...	15
7½	6	7½	...	...	...	...	...	...	...	12	...
8	5	8	...	...	...	...	...	...	...	4	...
8	6	8	...	...	...	...	1	8	17	21	72
8	6½	8	...	...	...	...	...	...	...	...	4
8	7	8	...	...	...	...	...	...	3	25	33
9	5	9	...	...	...	...	...	...	...	2	1
9	6	9	...	...	...	...	3	...	55	180	160
9	6½	9	...	...	...	...	...	...	...	22	34
9	7	9	...	...		...	...	9	6	38	49
10	7	10	...				...	2	1	5	3
10	8	10	...		...		...	1	3	9	3
12	6	12	...			...	...	...	...		1
Total thin edge or uniform			18	72	208	340	251	306	126	114	105
Total thickened edge.			0	0	0	0	4	122	108	356	418

Chap. XIV, p. 1053) and the lean bases generally develop transverse cracks at shorter intervals due to less tensile strength. On account of this tendency of rich mixes to aggravate surface cracks in the overlying asphaltic surfaces most engineers advocate a base mix of from 1:3:5 to 1:2½:5, either one of which is a good practical mix as regards requisite resistance to moisture absorption and practical construction manipulation.

Pavements of this type on rural highways are generally constructed with an integral concrete edging to hold the edge of the mix from side displacement. The extra depth of concrete along the edge is usually considered to give the necessary extra strength of edge as compared with interior-corner conditions; that is, the base depth is made uniform and based on the interior-corner classification where integral concrete edging is used. Where no edging is used the outer edge should be strengthened by any simple method, either extra edge depth or raised curbing.

In designing base depth for this type of pavement it is not permissible to figure on future strengthening. Adequate base strength must be provided as future renewals of worn-out asphaltic concrete surfaces remove the old surface course and replace with fresh mix; that is, resurfacing adds nothing to the final strength of this type of pavement.

The asphaltic surface wearing course varies, as a rule, from 2″ without a binder course to 3″, including a binder course. These depths are the result of experience and represent what appear to be feasible depths, considering construction and maintenance problems. The variation in depth of asphalt surfaces as between 2 or 3″ has little effect on the pavement base depth.

RECOMMENDED FACTOR VALUES FOR ROADS OF CLASSES I AND II IN DISTRICTS PERMITTING A 28,000-LB. GROSS VEHICLE ROAD

Design wheel load, including impact W, pounds	12,000
Load distribution factor J, interior corners	1.3
Design tensile stress, plain concrete base:	
1:3:6 mix, pounds	280 per square inch
1:2½:5 mix, pounds	320 per square inch
1:2:4 mix, pounds	360 per square inch
Soil-support factor P, ordinary soils	1.0
Soil-support factor P, gravel and macadam	0.9

These values result in the following recommended base depths for 28,000-lb. gross vehicle load. See Fig. 141 for more complete graphic results for different mixes and different design loads.

Ordinary foundation soils:

1:2½:5 mix

$$\text{Depth of base } d = 1.0\sqrt{\frac{1.3(12,000)}{320}} = 7''$$

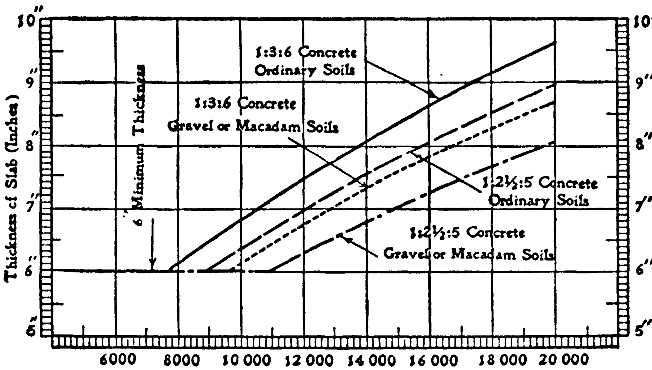
Gravel and macadam subsoils:

$$\text{Depth of base } d = 0.9\sqrt{\frac{1.3(12,000)}{320}} = 6.3''$$

A few examples of ordinary practice in base depths are given as follows. Table 85 (p. 424) shows recent city-street practice on heavy-traffic streets.

Illinois (rural highways)	7″	(1:2 :3½ mix)
Pennsylvania (rural highways)	6″	(1:2½:5 mix)
New Jersey (rural highways)	6″	(1:3 :5 mix)
New York (rural highways)	5″	(1:2½:5 mix)
New York City (business streets)	9″	(1:2½:5 mix)

Depth over Trenches.—Increase in base depth applies as discussed on page 416. For 7-ton truck loading it is probably desirable to increase for a 6″ standard base of 1:3:6 concrete for trenches over 3′ wide, for a 7″ depth of base where the trench width exceeds about 3.5 to 4′.



Design Wheel Load in lbs. (Includes Impact Allowance)

3½	ton truck (16,000 lbs. gross load)	Design Wheel load	8500 lbs.
5	ton truck (22,000 lbs. gross load)	Design Wheel load	10,000 lbs.
7	ton truck (28,000 lbs. gross load)	Design Wheel load	12,000 lbs.

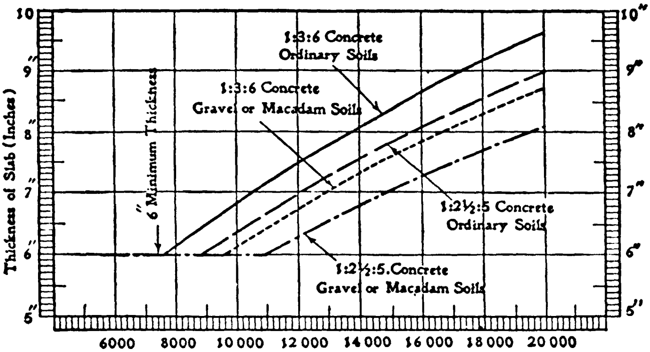
General Formula $d = P\sqrt{\frac{1.3W}{S}}$.

FIG. 141.—Theoretical depth of cement concrete bases under bituminous concrete surfaces.

Brick, Stone Block, etc., with Mastic Filler on Sand or Cement-sand Cushion and Concrete Base.—This design is rarely used on rural roads, as it is expensive. The advantage of the mastic filler lies in the prevention of surface cracks. The prevention of these cracks on the score of better appearance of the pavement is important in cities, but of secondary importance on rural highways. The disadvantage of the mastic filler lies in the loss of cantilever beam strength in the surface course and the production of more impact as the road serves traffic. It is not likely that the increase in impact is balanced by the cushion depth under the brick. It is certain that the base cannot be reduced in thickness below that required for sheet asphalt and the probabilities are that a slight increase is logical. Formula, $d = \sqrt{\frac{1.3W}{S}}$, with the additional impact allowance (see p. 404), seems to be about as good a guess as can be made on the basis of comparative action under traffic tests. This is perhaps a little ultraconservative.

For depths in use, see Tables 85 (p. 424) and 86 (p. 426). Figure 142 illustrates graphically the results obtained by applying this formula.

Grouted Brick, Stone Block, etc., on Cement-sand Cushion and Concrete Base.—These types have the advantage of considerable beam strength in the surface layer as well as the base beam strength; according to the Office of Public Roads tests previously quoted, they apparently do not, as a rule, develop as much beam strength as the monolithic form of brick pavement, but the difference is slight. The evidence of actual pavements indicates that they give satisfactory service with bases somewhat less in depth than the asphaltic concrete-top type. Personally, the author believes that



Design Wheel Load in lbs. (Include Impact Allowance)

3½ ton truck (16,000 lbs. gross load) Design Wheel load 10,000 lbs.
5 ton truck (22,000 lbs. gross load) Design Wheel load 12,500 lbs.
7 ton truck (28,000 lbs. gross load) Design Wheel load 15,000 lbs.

General Formula $d = P\sqrt{\frac{1.3W}{S}}$

FIG. 142.—Theoretical depth of cement concrete base under brick surface (mastic filler cement sand cushion).

the semimonolithic type will work satisfactorily under a design based on the depth of base $= \sqrt{\frac{0.9W}{S}}$; this is a rational modification of the asphaltic-type formula, as the grouted surface slab certainly adds more strength to the pavement than an asphaltic top and by beam action gives a wider distribution of load over base cracks and corners. As these formulas are intended to give relatively correct results, and as experimental results are meager, this form will be tentatively adopted. Thus the application of the corner load formula to this type is a speculative proposition.

The base under this formula becomes for loam and clay soils (28,000-lb. gross vehicle load):

$$1:2:4 \text{ mix } \sqrt{\frac{12,600}{360}} = 6''$$

$$1:2\tfrac{1}{2}:5 \text{ mix } \sqrt{\frac{12,600}{320}} = 6.3''$$

$$1:3:6 \text{ mix } \sqrt{\frac{12,600}{275}} = 6.8''$$

On gravel or old macadam subsoils the depths can be safely reduced ½ to ¾″, but should under no circumstances of either load or soil be reduced below a 5″ minimum. Figure 143 illustrates graphically the results of applying this formula.

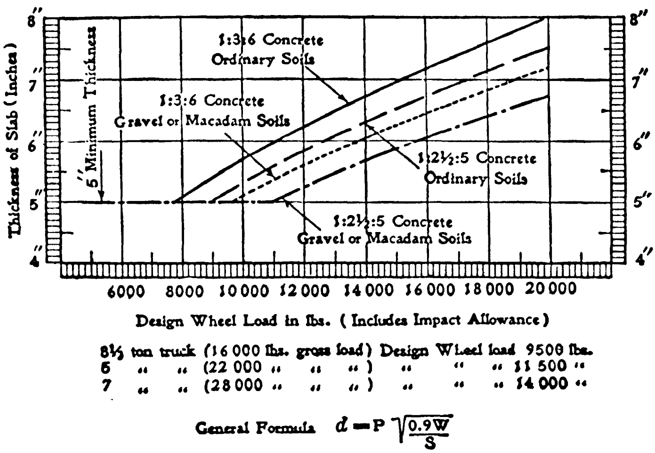


FIG. 143.—Theoretical depth of cement concrete bases under 4-in. brick surface (cement grout joints, cement sand cushion).

Monolithic-brick-pavement Slabs.—This type has the same general characteristics as the monolithic plain concrete as far as impact and load distribution at cracks are concerned. The difference between these two types lies in the fact that the plain concrete pavement is a true monolith and the combination of brick and concrete introduces a plane of weakness between the surface and bottom layers. The Bureau of Public Roads experiments indicated that the beam strength of the so-called monolithic-brick-pavement slabs was somewhat greater than the sum of the resisting moments of the upper and lower layers considered separately. Values of modulus of rupture for well-grouted brick surfacing slabs, as determined by the Bureau of Public Roads, range from 600 to 1000 lb. With these data, it is possible to make a speculative approximation of comparative strength depth. On page 412 the average depth of plain concrete 1 : 1½ : 3 mix using a tensile strength of 400 lb. per square inch was figured as 9.2″ for a design load of 14,000 lb.

The resisting moment of this beam per foot of width is approximately 67,000 in.-lb. It will be assumed that 75% of this must be handled by the sum of the resisting moments of the base and top considered separately, 50,000 in.-lb. Assume a working stress of 600 lb. for the brick top. A 4″ brick slab has a resisting moment per foot width of 19,200 in.-lb. This leaves 31,000 in.-lb. for the base to handle.

Expressed as a formula, the depth of concrete base = $\sqrt{\frac{1.8W - 9600}{S}}$ for loam and clay soils. On gravel or old macadam the depth can be reduced about an inch but under no circumstances of load or soil should a depth of less than 4″ be used.

Depth of top	Average depth of concrete base, inches	Total depth of pavement, inches
4″ brick top:		
1 : 1½ : 3 mix. . . .	6.3	10.2
1 : 2 : 4.	6.7	10.6

A few examples of current practice in monolithic-brick construction follow: Monolithic-brick construction is not in much favor in the northern states on account of cracking due to frost action and slab warping. Central longitudinal joints are not generally used on this type.

	Top, inches	Base, inches	Total, inches
Illinois.	4	(1 : 2 : 3½ mix)	8
Indiana.	4	4	8

Current Practice (Two-course Pavements).—Table 85 records current practice (1922) in two-course pavement construction on heavy-traffic city streets.

TABLE 85.—CURRENT PRACTICE IN DEPTH OF CONCRETE BASE UNDER PAVEMENTS ON HEAVY-TRAFFIC CITY STREETS (1922)

City	Type of surface				Remarks
	Asphaltic concrete on concrete base, inches	Brick or stone block cement grout filler, inches	Brick or stone block mastic filler, inches	Concrete, inches	
Cleveland, Ohio. . . .	8	6	7	. . .	Tendency towards 7 to 8″ except for grout-filled block.
Detroit Mich. . . .	8	8	8		
Chicago, Ill.	8	8	8	. . . .	This is recommended future practice.
Philadelphia, Pa. . .	6	6	6		
Indianapolis, Ind. . .	6	6	6	. . . .	Gravel subsoil
Baltimore, Md.	6–8	6–8	6–8	. . . .	8″ on poor soil.
New Orleans, La. . .	6	6	6		
Boston, Mass. . . .	6	6	6	. . . .	Has been satisfactory.
Portland, Ore.	6	6	6	6–10	

Effect of Statutory Load Limit on Thickness of Rigid Pavements and Concrete Pavement Bases.—Table 86 gives a quick means of comparing the effect of load limit on rural pavement design. The black-faced type indicates the recommended depths for the statuatory gross vehicle limitation of 28,000 lb. which seems to be gaining favor as a practical basis of traffic regulation for the main roads. The depths given approximate equal strength for the different types and can be used as the basis for comparative estimates of cost. The determination of the effect of statuatory load and equal strength for different types are the main practical advantages of the foregoing analysis.

SPECIAL QUALIFICATIONS AND DRAWBACKS OF DIFFERENT PAVEMENTS

Limitations Imposed by Steep Grades.—The following tabulations[1] represent current practice in regard to the steepest advisable grades on which various types are satisfactory, considering safety and maintenance costs.

Surface Material	Per Cent
Wood block[1]	3
Asphalt block[1]	6
Brick[1]	10
Sheet asphalt[1]	5
Asphaltic concrete[1]	7
Bituminous macadam (seal coat)[1]	8
Bituminous macadam (no seal coat)[1]	10
Cement concrete[a]	8
Hillside brick block[1]	12
Stone block[1]	12
Wooden block[2]	2
Asphalt block[2]	4
Brick (grout joints)[2]	5
Brick (mastic joints)[2]	8
Concrete[a]	5–7
Bituminous macadam with flush or squeegee coat (in sandy country, 6% when coarse sand is sprinkled on surface)[2]	5
Bituminous macadam without squeegee[2]	8
Water-bound macadam[2]	8
Hillside brick[2]	12
Stone block with open joints[2]	12

[a] Hard to construct on grades over 5 % (special care needed).
[1] Taken from Agg's "Roads and Pavements."
[2] Taken from Harger's "Rural Highway Pavements."

On steep grades, stone block seems to be the best solution, hillside brick second, penetration one coat pour bituminous macadam third, and water-bound macadam fourth. The last two become slippery if maintained by surface oiling and it has been necessary in some cases to build a specially wide shoulder treated

TABLE 86.—SUMMARY OF RECOMMENDED THEORETICAL DEPTHS OF CEMENT-CONCRETE BASES FOR DIFFERENT PAVEMENTS UNDER DIFFERENT MAXIMUM LOADS ON DIFFERENT SOILS (IN INCHES)

(The last column gives prevailing practice in base depth for each type, 1919 to 1922.)

Pavement	Recommended depths of cement-concrete bases based on modified corner load formula $d = P\sqrt{\frac{JW}{S}}$						Current practice 1919–1923, inches
	3½-ton truck 16,000-lb. gross load		5-ton truck 22,000-lb. gross load		7-ton truck 28,000-lb. gross load		
	Ordinary subsoils, inches	Gravel or macadam, inches	Ordinary subsoils, inches	Gravel or macadam, inches	Ordinary subsoils, inches	Gravel or macadam, subsoils, inches	
Plain concrete:							
1 : 1½ : 3 mix	7.2 & 7.8	6.5 & 7.0	7.9 & 8.8	7.2 & 8.0	8.7 & 9.7	8.0 & 8.8	6–10
1 : 2 : 4 mix	7.6 & 8.2	6.9 & 7.4	8.4 & 9.2	7.6 & 8.3	9.2 & 10.0	8.4 & 9.1	
Reinforced concrete (mesh and bar): Central longitudinal joint 1 : 1½ : 3 mix	6.3 & 6.9	6.0 & 6.4	6.7 & 7.5	6.2 & 6.8	7.3 & 8.2	6.5 & 7.3	
Reinforced concrete (corner and exterior tie bars only):							
Central longitudinal joint 1 : 1½ : 3 mix	6.5 & 7.0	6.0 & 6.4	7.0 & 7.8	6.3 & 7.1	7.7 & 8.5	7.0 & 7.7	5– 9
1 : 2 : 4 mix	6.9 & 7.4	6.4 & 6.8	7.4 & 8.2	6.7 & 7.4	8.1 & 9.0	7.4 & 8.1	
Monolithic brick, 4″ brick on 1 : 1½ : 3 concrete base	4.5	4.0	5.0	4.5	6.0	5.5	4.0

2″ to 3″ asphaltic concrete on concrete base:							
1 : 3 : 6 mix	6.5	6.0	7.0	6.3	**7.5**	**6.8**	**5– 8**
1 : 2½ : 5 mix	6.0	6.0	6.5	6.0	**7.0**	**6.3**	
2″ asphalt block on cement-concrete base:							
1 : 2½ : 5 mix	6.5	6.0	7.0	6.3	**7.5**	**6.8**	**5– 8**
4″ brick (mastic joint filler) on cement-concrete base:							
1 : 3 : 6 mix	6.8	6.2	7.6	6.9	**8.3**	**7.5**	**6– 8**
1 : 2½ : 5 mix	6.5	6.0	7.0	6.3	**7.8**	**7.1**	
4″ brick (cement-grout filler):							
1 : 3 : 6 concrete base	5.5	5.0	6.1	5.5	**6.7**	**6.2**	**5– 8**
1 : 2½ : 5 concrete base	5.2	5.0	5.7	5.2	**6.3**	**5.7**	
5″ stone block: cement grout filler							
1 : 2½ : 5 concrete base	5.2	5.0	5.7	5.2	**6.3**	**5.7**	**5– 7**

NOTE: Where two depths are given the smaller is for interior areas and the larger for exterior edge depths. Where one is given it applies to interior areas.

Typical recommended pavement sections showing details are given in Chap. VI under the discussion of each type of pavement. The object of this table is to give a tentative basis of estimating the amount of materials and the cost per square yard for the different types suitable for different maximum load conditions.

Table 86 in conjunction with Table 74, page 391, provides a means of computing reliable comparative cost estimates for all ordinary standard types based on equal strength.

with gravel or stone for horse traffic. It is very difficult to get a satisfactory surface on concrete on grades over 5 to 7%.

Classification for Safety of Traffic.—The sheet asphalts and similar constructions are dangerous for high-speed traffic even on fairly level grades during sleet storms or light rains and are not recommended for roads outside of villages except for reconstruction where the large saving due to the use of this type overcomes the factor of reduced safety. Slipperiness can be partially overcome by the use of a larger percentage of small stone in the mix, such as Topeka, Amiesite, Warrenite, etc.

Bituminous macadams, concrete, brick, stone block, water-bound macadams and small stone or brick cubes can be ranked as safe surfaces for high-speed traffic.

Recommended Types.—Bituminous macadams are recommended for Class II traffic and resident village streets; water-bound macadam for Class III traffic; concrete for Class I traffic outside of villages; brick for village business streets; stone block for hills on Class I traffic; asphalt block for extremely heavy Class I traffic. Sheet asphalt, Topeka, etc., are to be avoided for original construction where traffic travels at high speed, but its use for reconstruction has decided advantages from an economic standpoint. Its most suitable location is a resident village or city street or for heavy, slow traffic.

Value of Smooth-riding Quality of Pavements.—This phase of type selection and maintenance limits of expenditure is given in Chap. VII (p. 545).

Failures.—The common causes of failure of different pavements due to structural defects are as follows. The details of inspection are taken up in Chap. XVI.

Stone Block.—Failures rare; will stand lots of abuse in construction.

Asphalt Block.—Failures rare. When they occur due to poor block.

Water-bound Macadam.—Failures rare. When they occur are generally due to poor rock, small-sized stone in top courses, and insufficient rolling or puddling.

Penetration Bituminous Macadam.—Failures not uncommon due to the use of too much soft binder, unequal application, and overheating of binder. The asphalt companies advocate the use of too much bitumen.

Concrete.—Failures not uncommon due to inferior materials, particularly dirty sand, and to poor manipulation, weak mix, and too much water content.

Brick.—Failures not uncommon due to poor brick and careless grouting.

Sheet Asphalt and Topeka Mix.—Failures not uncommon due to overheating and poor mix.

DETAILS OF DESIGN AND CONSTRUCTION OF HIGH CLASS PAVEMENTS

STONE AND GRAVEL FOUNDATION COURSES FOR FLEXIBLE-TYPE PAVEMENTS

Preparation of Subgrade.—It is evident from the pressures to which a road is subjected that the subgrade must be well consolidated before placing the foundation stone. This is usually effected by rolling with a 10- or 15-ton steam roller, exerting a pressure of 350 to 500 lb. per linear inch or wheel width, and is continued until the grade is firm and compact.

The difficulties of consolidation in different soils and the methods of overcoming them are discussed under Inspection (pp. 1289 and 1291).

Kinds of Foundation Courses.—The foundation courses in ordinary use are as follows:

1. Crushed stone.
2. Screened gravel.
3. Field-stone subbase.
4. Pit-gravel subbase.
5. Field-stone subbase bottom course.
6. Pit-gravel subbase bottom course.
7. Quarry-stone or Telford base.

1. Broken-stone Bottom Course.—This style of construction is the one in most general use. Where local stone is abundant and well distributed, such a course will cost[1] from $3.50 to $5 per cubic yard rolled in place; where imported stone is necessary, the cost depends largely upon the freight rate and the length of haul and may run as high as $7. Bottom of this kind is generally used where the total depth of stone metaling does not exceed 6 to 8″ after rolling. Beyond these depths it is often cheaper to substitute subbase or subbase bottom course for a part or the whole of the broken-stone course. (See page 390.)

The method of construction by the New York State Highway Commission is shown in extract from Specifications on page 1434.

Where imported stone is specified or the local stone is suitable for both top and bottom courses, the size used for bottom course ranges from 2¾ to 3¾″ in its greatest dimension; the smaller-sized crusher output is used for the top course, for concrete, and for filler; where the local material is only fit for bottom, the course is made up of stone ranging from 1 to 3¾″ in order to use up the total output of the crusher. The stone smaller than 1″ is used for filler, on the shoulders, and sometimes for the cheaper grades of concrete. In specifying the size of stone for a particular job, economy is considered. Stone of sizes from 1 to 3¾″ is perfectly satisfactory. The only reason for limiting the usual size from 2¾ to 3¾″ is that this practice leaves the 1 to 2¾″ stone for the top course; a uniform grade is important for the top and the size mentioned gives a smooth finish.

[1] All costs are for general comparative purposes and are based on 1926 cost conditions.

The ratio of loose depth to rolled depth is approximately 1.3 to 1.

Where filler is not used in the construction of the bottom course more binder is required for the top; it is probable that the use of filler is the better construction, but it must be of good quality.

The clause concerning teaming in the quoted specifications is a dead letter. Teaming helps to consolidate the bottom, provided it is distributed over the full width and the course watched to prevent loss of shape when the traffic is first turned on or after a long-continued rainfall.

2. Screened-gravel Bottom Course.—Screened gravel 1 to 3½″ in size is used in place of crushed stone; the course is constructed in the same manner as described above, except that a filler containing some clay or clay loam is preferable to a clean sand, and it is often necessary to wet the course in order to consolidate it satisfactorily. It is also advisable to apply a small part of the filler before the course is rolled.

A gravel bottom should be made somewhat thicker than a crushed-stone bottom, as the fragments do not interlock so firmly as crushed stone.

Gravel is suitable for Class III or IV traffic and the choice between screened gravel or crushed stone depends entirely on relative cost. Gravel-bottom courses are not advised for traffic of Classes I and II. Under favorable conditions a screened-gravel bottom course will cost from $2 to $3 per cubic yard, rolled in place. A coarse pit-run gravel is preferable to a screened-gravel bottom. A typical specification is given on page 1434.

3. Field-stone Subbase.—Field-stone subbase is constructed, as shown in the cut, of field boulders roughly placed, the intervening spaces being filled with gravel, waste ¾″ stone, or stone chips.

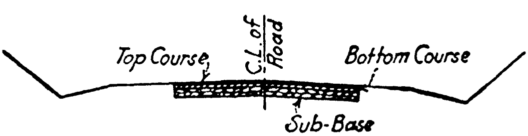


FIG. 144.—Boulder subbase.

No attempt is made to finish the top of the course exactly to line and grade, as any small inequalities can be filled with bottom stone. This type of subbase is suitable for all classes of traffic. The depth varies from 5 to 20″, depending on the soil encountered, traffic the road carries, and also on the size of the available field stone, but for depths of over 8″ the foundation must be constructed as a series of layers not exceeding 8″ each, in order to get proper compaction and filling. In designing a bottom course of this kind, care must be taken to have accurate data as to the average size of stone available. If the demands of a foundation were fully satisfied by a 5″ subbase course, it might still be more economical to use a 7″ course if the stone averaged 7″, because the extra work of sorting and sledging to a 5" size would result in a higher cost per square yard than for a 7" depth.

The amount of stone and filler required per cubic yard in place is approximately as follows:

Loose depth, inches	Rolled depth, inches
6	6
10	8
15	12

This type of base requires approximately 1/3 cu. yd. of gravel or 3/4" stone for filler per cubic yard finished subbase course.

Under favorable conditions this subbase can be constructed for $1.80 to $3 per cubic yard (1922 cost conditions). Specifications are given on page 1425.

4. Pit-gravel or Creek-gravel Subbase.—Stony gravel is satisfactory material for subbase; it can be readily constructed for any depth from 2 to 24" as required, and where a pit or creek bar is near, the cost of such a course should run from $1.50 to $2.50 per cubic yard. However where the depth is greater than 5" it must be constructed and compacted in a series of layers no one of which exceeds 5" in depth.

The ratio of loose to consolidated gravel for such a course is approximately 1.2 to 1.

5. Field-stone Subbase Bottom Course.—Subbase bottom course is essentially the same construction as subbase, except that, as

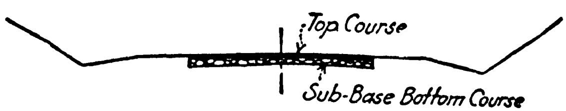


FIG. 145.—Boulder subbase bottom course.

the top course is placed directly upon it, the stone must be more carefully assorted as to size, more carefully placed as to line and grade, and a better grade of filler must be used. Subbase bottom course is advised only under traffic of Classes III and IV. Traffic of Classes I and II requires a middle course (see p. 390).

Crushed stone (crusher run) or coarse gravel makes a satisfactory filler.

The course can be of any depth from 5" up, depending, as for subbase, on the soil and average size of stone; it is practically impossible to make a large stone bottom of this kind conform exactly to line and grade; a variation of 1" either above or below grade is usually allowed and the inequalities taken out by the top stone; this requires that the top course must be at least 3 to 4" deep after rolling.

Subbase bottom is especially applicable for traffic of Classes III and IV on long stretches of road, requiring a depth of 9 to 20″; depths of over 9″ must be constructed in successive layers. It usually costs from $2 to $4 per cubic yard in places where fence stone is available, and by its use the item of higher-priced stone is reduced. On hard foundation soils, however, it is generally better to use 4 to 5″ of ordinary broken-stone bottom course instead of the subbase bottom course even if it is more expensive, because the small-stone construction is more uniform in its resistance to heavy loads and the top course will wear more evenly and longer.

An extract from the 1920 New York State Specifications is given below:

SUBBASE BOTTOM COURSE

"When field or quarry stone is used for constructing the foundation course it shall be of a hard, sound, and durable quality, acceptable to the engineer; the stones shall be placed by hand so as to bring them in as close contact as possible. All stone must be rehandled in placing. Dumping from wagons and leveling off the piles will not be permitted. When quarry stones are used they shall be placed on edge. The depth of single stone shall in no case be greater than the depth specified for the course, the width shall not be greater than the depth, nor more than 9″, and the length shall not be greater than one and one-half times the depth, nor more than 12″. The distribution of the stone shall be of a uniformity satisfactory to the engineer. The long dimension shall always be placed crosswise the road. After laying, this course shall be thoroughly rolled with an approved roller weighing not less than 10 tons, and shall then be filled with stone or coarse gravel as directed and again rolled until the stones are bound together and thoroughly compacted: but no gravel shall be used for filling except under written permission of the engineer. All holes or depressions found in rolling shall be filled with material of the same quality and the surface shall be rolled until it conforms to the lines and grades shown on the plans. When field stone is used, approved tailings may be used for filling. In all cases a sufficient amount of fine material (coarse sand or crusher screenings) shall be used to fill all voids. In limited areas where the use of a roller is impracticable, heavy tampers may be used to consolidate the material."

6. Pit-gravel Bottom or Subbase Bottom.—A stony gravel containing not over 15% of loam makes a satisfactory course; the depths vary from 4 to 18″; pit or creek gravel, even when unusually coarse, has from 40 to 60% of fine material; a suitable gravel for pit-run bottom should not contain more fine material which will pass a ¼″ screen than coarse material which will be retained on a ¼″ screen. If there is a large excess of fine, the gravel should be screened and remixed at the bin in proper proportions. Pit-run-gravel bottom should be used only for traffic of Classes III or IV.

The great difficulty in this construction is to get proper consolidation without too much delay. It is advisable to lay a course of this kind at least 2 weeks ahead of the top stone so that traffic and rains may help consolidate the course. The addition of 10% of loam to clean gravel will quicken the consolidation. This can be done either at the pit when stripping, by leaving a thin layer of loam, which runs down with the gravel in loading, or by placing from ½ to 1″ of loam on top of the gravel as it is spread on the road. The author has succeeded in getting rapid consolidation by snatching loaded teams over the loose course with the road roller; the roller continually smooths out the gravel and eases the haul for the teams;

the horses' hoofs and wagon wheels punch into the gravel and pack it down rapidly; a traction engine with lugs on the wheels is also effective in place of a standard roller. Sprinkling helps. A gravel bottom consolidates unevenly and it is always necessary to reshape it somewhat after consolidation; about 5 cts. per cubic yard should be allowed for this reshaping of crown and elimination of humps and hollows.

A properly consolidated gravel bottom will permit a 4-ton load on 3½" tires to pass over it without making a wheel mark over ⅛" deep; this is a simple construction test. This construction has been discussed in some detail as it is the most economical type of bottom in a large number of cases, but it is not generally favored because it is harder to consolidate than the other types of bottom. With a 3" or, preferably, a 4" macadam top it has proved satisfactory on Class III and IV roads.

The cost of a gravel bottom ranges from $1.20 to $2.20 per cubic yard in place, provided the hauls are short.

The depths of gravel are gaged by blocks or lines and the ratio of loose to rolled depth is approximately 1.2 to 1.

7. Telford Base.—Telford base is rapidly going out of use in the United States because of the difficulty of maintaining a top course laid upon it. It seems to be too rigid and is more expensive than subbase or subbase bottom course, costing about $3 to $5 per cubic yard under favorable conditions.

A good description of a Telford construction is given by William Pierson Judson, in "Roads and Pavements." The following quotation is from his book:

> "On this subgrade are then placed by hand the stones forming the Telford foundation, which may vary in size as shown below; each stone must be set vertically upon its broadest edge, lengthwise across the road and forming courses and breaking joints with the next course, so as to form a close and firm pavement. The stones are then bound by inserting and driving stones of proper size and shape to wedge the stones in their proper position. All projecting points are then broken with a sledge or hammer so that no projections shall be within 4" of the finished grade line.
>
> "The Telford foundation is then rolled with a steam roller of 10 or more tons' weight, until all stones are firmly bedded and none move under the roller. All depressions are then filled with stone chips not larger than 2½", and the whole left true and even and 4" below the line of finished grade and cross-section.
>
> "A good workman will average about 20 min. in setting a square yard of this Telford foundation, which may be formed of any kind of quarried rock which is most available."

The practice in 1901 in four states was as follows:

TABLE 87.—SIZES OF STONE FOR TELFORD FOUNDATION, IN INCHES

State	Depth as set on edge		Width as set		Length set across road		Remarks
	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum	
New Jersey	8	8	4	..	10	..	Alternate end stones, double length
Massachusetts	6	5	10	4	15	6	2" gravel rolled on subgrade as base
Connecticut	8	8	10	6	18	8	Macadam covering formed in one layer
New York	8	6	10	4	15	6	Used only on unstable ground as foundation for macadam

TELFORD BASE—SPECIFICATIONS (STATE OF CONNECTICUT)

"95. **Description.**—This base course shall consist of a foundation for the surface course or pavement, 8" in depth unless otherwise specified, constructed on the prepared subgrade, of large stones wedged in place by the addition of smaller ones, in accordance with these specifications.

"96. **Materials.**—The materials for this work shall consist of approved, sound, tough, durable stones, free from clay, loam, or other foreign substances. The pieces shall be approximately rectangular in section, having a depth of 8" after knapping, a width of from 6 to 10", and a length of from 8 to 18". The small stone for filling the voids in the large material shall consist of material at least equal in quality to that of the large stone.

"97. **Construction Methods.**—The Telford stone shall be laid in courses, by hand, with the broad edge down and with the long dimension at right angles to the center line of the roadway. They shall be laid perpendicular to the finished surface in close contact, breaking joints, and shall be wedged in their correct position by inserting and driving, in all places where practicable, small stone of suitable size. All projecting points shall be napped off and the remaining voids filled with small stones, so that the finished surface shall be true and uniform. This base course shall be rolled with a three-wheel power roller, weighing not less than 10 tons, until compacted satisfactorily and true to the grades and cross-sections given. The filling and rolling shall follow the laying of the large stone closely. This course shall not be constructed more than 1000 lin. ft. in advance of the surface course or pavement, unless otherwise permitted by the engineer.

"If at any time, the subgrade material should become churned up or mixed with the Telford stone, the contractor shall, without additional compensation, remove the mixture, reshape and compact the subgrade, and replace the materials removed with clean Telford stone, which shall be rolled and filled until compacted satisfactorily and uniform with the surrounding surface.

"98. **Basis of Payment.**—This week shall be paid for at the contract unit price per square yard for '8" Telford base course' complete in place, which price will include all materials, equipment, tools, labor, and work incidental thereto."

Distribution of Stone in Foundations.—On light-traveled single-track roads most of the traffic normally keeps to the middle 10 to 12'. It would therefore appear logical to make the central portion of the road thicker than the sides. This applies without doubt to roads of moderate traffic where the vehicles generally travel in the center of the macadam and only occasionally turn out to pass, but for heavy-traffic double-track roads the assumption is wrong. On such roads the greatest wear and the heaviest wheel load occur about 1′ from the edge of the hard pavement and many of these roads develop the shape shown in Fig. 146. It therefore seems advisable to keep the full depth of metaling for the full width on traffic roads of Classes I, II, and III.

For Class IV traffic the varying thickness indicated in Fig. 147 is applicable.

FIG. 146.—Shallow ruts along edge of Class I traffic macadam road.

Special Foundation Problems.—Long stretches of comparatively level ledge rock, muck, and vegetable loam may be placed under this head.

Where a road is on the surface of ledge rock for any distance, the usual cross-section of part cut and part fill cannot be used because of the high cost of shallow rock excavation for ditches; the grade should be lifted to make the normal section fill and the best available material (not clay) used in its construction. Where conditions of this kind prevail, dirt is usually hard to obtain and often a stone fill is cheaper and also more satisfactory.

The construction shown (Fig. 148) was used for a stretch of 2½ miles on the Le Roy-Caledonia State Highway in New York, where

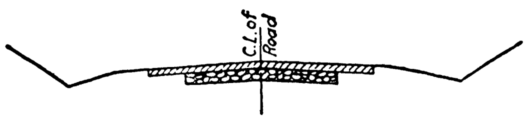


FIG. 147.—Mushroom type of foundation Class IV traffic roads.

ledge rock was encountered as described. The price for the stone fill was \$1.23 (1910) per cubic yard in place, constructed as shown; the road was built in 1910 and has given satisfaction; the minimum thickness of top for such a fill is 3 to 4″, as it is impossible to construct it exactly on line and grade; it was found that, by allowing a variation of 1″ either above or below the grade elevation, the fill could be readily constructed, and these small inequalities were taken out with the top stone. A top course having such a variable thickness should be paid for by weight and not by volume in place.

Peat, Muck, Vegetable Loam, or Silt.—Where the material is semifluid, the only solution is a pile and grillage foundation.

Swamps, as ordinarily encountered, can be treated successfully by using a corduroy or mattress foundation covered with a deep fill of gravel or large stone. In some cases where the muck is should be a separate item separately paid for. The filler for boulder or Telford base should be hard, coarse gravel, 3/8" crushed stone, or stone chips.

Macadam foundation failures are due to the same cause as concrete failures or brick failures or any other failure—*ignorance and carelessness*. There is no such thing as type failure.

MACADAM SURFACE COURSES

Water-bound Macadam Pavements

Water-bound macadam surface courses are constructed of crushed fragments of suitable rock filled with rock dust and sprinkled and rolled until firm and hard. All types of gravel, boulder, and stone bases are used as described in the first part of this chapter. Standard Specifications are given on page 1454.

Water-bound macadam pavements are satisfactory and economical under light traffic up to about 800 vehicles daily, provided they are systematically maintained and surface ravelling prevented by applications of calcium chloride or light bituminous surface treatments. They have been used successfully under quite heavy traffic, 2000 to 3000 daily, but for a volume of over 500 to 800 daily the maintenance difficulties generally result in the selection of a bituminous-macadam penetration type. The essential advantage of the water-bound type lies in low first cost, simplicity of construction, ease of repair, and safe and easy-riding qualities if the road is well constructed and maintained. Under normal agricultural-district traffic of less than 500 vehicles daily, the final cost of this type of pavement, including yearly maintenance and renewal, is distinctly lower than higher types of surface. When well designed and constructed of adequate thickness (see pp. 390 to 391), they can be used effectively as a base for higher-type surfaces. Water-bound macadam is a very satisfactory type for local town or country roads and for Class III traffic on primary state systems.

Water-bound macadams, including base and top, range in initial construction cost[1] from $1.50 to $3 ($1.90 average) per square yard, depending on character of foundation soil, traffic, and local material supply. The maintenance charge will range from 3 to 10 cts. per square yard, depending on varying conditions; and the renewal charge distributed over the life of the surface will probably average about 5 cts. for Class IV traffic to 11 cts. for Class II traffic per square yard per year, 1922 cost conditions. A surface life of 6 to 12 years is reasonable for this type under a traffic of 300 to 800 daily (10-hr. count) if the road is fairly well maintained, although a considerable number of cases do not fall within these limits (see Chap. VII).

The use of fairly good materials is desirable, but the high standards required for heavy-traffic roads should not be applied to this type, as needless cost is to be avoided under light-traffic conditions. Materials, details of design, and inspection details are discussed later.

[1]1926 cost conditions Western N. Y.

The following recommended designs can be used with assurance of reasonably good results.

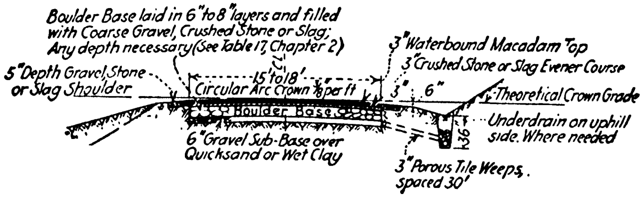


FIG. 150*A*.—Recommended waterbound macadam pavement section boulder base construction Class II traffic. For discussion of crown see page 443.

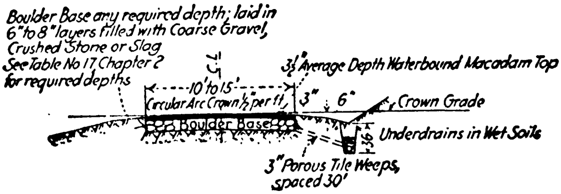


FIG. 150*B*.—Recommended waterbound macadam pavement section boulder base construction Class III or IV traffic.

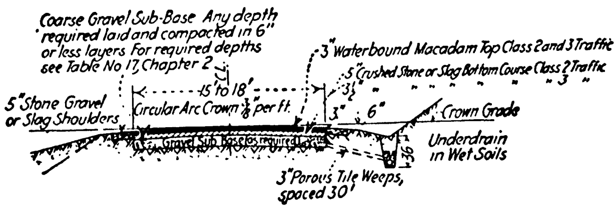


FIG. 151*A*.—Recommended waterbound macadam pavement section gravel foundation Class II or III traffic.

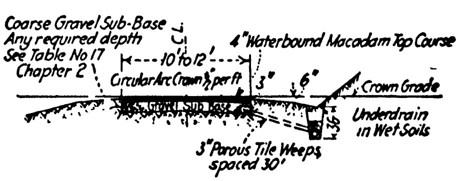


FIG. 151*B*.—Recommended waterbound macadam pavement section gravel foundation Class IV traffic.

Recommended Designs.—The accompanying recommended designs represent conservative practice; a large mileage of thinner pavements has been built and has served for a short time with moderate satisfaction; as a matter of fact, no well-constructed macadam road having a depth of at least 80% of the values recommended can be a serious failure, as it can be easily strengthened in the future by the addition of stone depth. It, however, seems undesirable in most cases to resort to resurfacing in too short a period, and for this reason caution should be exercised in reducing the depths shown in districts permitting a legal gross vehicle load of 28,000 lb. (see Figs. 150 and 151, p. 439).

Suitable Materials. *Surface-course Coarse Aggregate.*—Any tough, hard-crushed stone which breaks in roughly cubical form and does not air or water slack is suitable for coarse aggregate. Limestone, granite, gneiss, trap, and hard sandstones are generally satisfactory. For Class III traffic (300 to 800 daily) it is desirable to use a stone having a French coefficient of hardness of 7 or better (6% of wear or less) (see Chap. XI for tests). Stone as soft as 4 French coefficient has been used with moderate success, but where a soft stone is necessary because of the prohibitive cost of a better grade the size should be increased. The usual size of coarse aggregate for top course is 1¼ to 2½″; for soft stone the size should be 2½ to 3½″, using the smaller grades for the foundation course. For purely local roads carrying less than 300 vehicles daily practically any available local stone which does not air or water slack can be used. Crushed slag is suitable under light traffic. Crushed slag is not advocated for top course on the heavier-traffic roads. Crushed slag should weigh at least 1800 lb. per cubic yard loose measure and show a per cent of wear not exceeding 12 by the standard slag abrasion test.

Surface-course Binder.—Limestone screenings make the best filler, as they have considerable cementing quality and bond the stone well. Where the coarse aggregate is trap, granite, or sandstone, it is difficult to get a good puddle, and on the more important roads at least 50% of limestone dust is mixed with the other screenings. Surface oiling tends to minimize the necessity for limestone screenings, but the authors prefer at least 50% limestone dust on water-bound macadam roads on primary state systems. For purely local agricultural roads, the natural rock screenings are used.

Oiling.—There are a number of light asphaltic oils and refined tars on the market which serve well for surface applications. The quality of these bituminous materials is discussed under Maintenance, Chap. VII. Specifications are given in Part III. Personally, the author prefers a good cold-tar product to asphaltic oils for surface application, as they seem to have better penetration and to harden quicker. They are probably a little more slippery and perhaps not quite so durable, but, on the whole, tar is superior to asphalt for surface treatment of water-bound macadam roads. For surface oiling of penetration bituminous macadam, it is probably best to use asphaltic oil where the pavement binder is asphalt and to use the cold, light tar where the pavement binder is a heavy tar.

Macadam Bottom Courses, Coarse Aggregate.—For crushed-stone bottom courses, any stone which will stand the necessary rolling to compact and which does not water slack is suitable. Crushed slag weighing not less than 1800 lb. per cubic yard loose makes an excellent material. No maximum per cent of wear is recommended. The cheapest available source is generally used, but it is necessary to decide what source will be used and to set the maximum per cent of wear for any specific contract to prevent argument and the use of needlessly inferior material. The size usually specified is 2½ to 3½″, but this is merely to permit the use of the 1¼ to 2½″ size for top, and a perfectly satisfactory bottom can be constructed of well-mixed stone ranging in size from ¾ to 3½″, provided pockets of fine and coarse stone are not permitted. Tailings can be used if knapped to the proper size.

Macadam Bottom Filler.—Coarse sand, stone, or slag screenings make the best filler; sandy loam is satisfactory on light-traffic roads. Filler is very important; it should be a separate item, separately paid for, and no material which softens when wet should be used, even for unimportant roads. *A considerable number of macadam failures are directly traceable to soft filler.*

Boulder-base Coarse Aggregate.—Any ordinary boulder or quarry stone is suitable provided it is not rotten. Large boulders must be sledged to suitable size. Maximum size is usually placed at 8″ for any dimension, with the depth thickness not greater than the finished depth of the course (see Specifications, p. 432).

Boulder-base Filler.—Crushed stone or slag or coarse, hard gravel containing not over 15% of loam is satisfactory. Fine sand or loam should never be used.

Gravel Bottom and Subbase.—A coarse, hard gravel containing not over 15% of loam and having a grading in size of at least 50% retained on a ¼″ mesh screen makes an excellent base. If the pit run is non-uniform and excessively fine, it should be screened and remixed in the proper proportions of fine and coarse.

Amount of Material Required per Cubic Yard of Consolidated Macadam.—*Top course* requires approximately 1.3 cu. yd., loose measure, of crushed stone (1¼ to 2½″ size) and approximately 0.5 cu. yd. stone screenings for binder and wearing course. The weight of this material will vary according to the kind of stone used. Commercial plants will furnish data of loose-measure weights of their standard sizes. Table 88 gives a fairly close means of estimating weight.

In western New York, for limestone having a specific gravity of approximately 2.7 (170 lb. per cubic foot), the total weight of stone and screenings per cubic yard of finished water-bound macadam top course, including the wearing course, is close to 4400 lb. From 50 to 80 gal. of water are required to puddle a cubic yard of top course.

Macadam bottom course requires approximately 1.3 cu. yd. loose measure of crushed stone, slag, or screened gravel ranging in size from 2½ to 3½″ and approximately 0.35 cu. yd. of sand or stone screening filler. Where the bottom is constructed of mixed sizes from 1¼ to 3½″, it requires approximately 1.25 cu. yd. loose per consolidated yard of finished macadam.

TABLE 88.—WEIGHTS[1] OF MATERIALS

Kind	Specific gravity	Broken stone, pounds per cubic yard	
		Loose spread 45% voids	Compacted 30% voids
Trap	2.8	2590	3300
	2.9	2680	3420
	3.0	2770	3540
	3.1	2870	3650
Granite	2.6	2400	3060
	2.7	2500	3180
	2.8	2590	3300
Limestone	2.6	2400	3060
	2.7	2500	3180
	2.8	2590	3300
Sandstone	2.4	2200	2830
	2.5	2310	2940
	2.6	2400	3060
	2.7	2500	3180

[1] "Asphalt Association Pocket Reference."

Pit-run gravel subbase requires approximately 1.2 cu. yd. loose measure per consolidated cubic yard.

Field-stone boulder subbase requires 1 cu. yd. of boulders per cubic yard of finished course where only one layer of uniform-sized boulders is used. Such a base requires 0.5 cu. yd. of gravel or crushed-stone filler. Where the depth of the base is greater than the average-sized boulder, approximately 1.25 cu. yd. of boulders are required per cubic yard of finished base, with from 0.3 to 0.4 cu. yd. of gravel or crushed-stone filler.

Design.—The design of macadam pavements considers the proper use of local materials, the depth required on different soils, width, crown, etc. Pages 366 to 392 developed the detail theory of total macadam depth under different traffic and soil conditions. Table 74 (p. 391) gives these depth data. Having established the total depths, alternate estimates are made on the basis of utilizing gravel, boulder, or crushed-stone base courses. The variations in depth of the macadam surface on these different bases were developed on page 390. The most economical combination is adopted (for example, see Official Report, p. 1095). Where boulder and gravel are both available, it is generally better and more economical to use boulder base, gravel filled, with a thinner macadam surface. While it is desirable to keep the depth of top course to a minimum on account of the high cost of this part of the pavement, it is not safe to use less than a 3" consolidated depth on account of the tendency to ravel and kick out.

Widths.—Widths were discussed in the chapter on Sections (pp. 130 to 142).

Crown.—A circular-arc crown is a good form. For the narrow single-track local roads, a total crown of 3/8 to 1/2" per foot of half width is a good rule. For the wider roads 16 to 18' and for village streets, the crown should be about 5/16 to 3/8" per foot on account of future use of the macadam as a base for asphaltic concrete or small block surfacings. One-way banking on sharp curves is desirable for rural roads carrying over 300 vehicles daily, but is a useless refinement on light-traffic local roads.

See Chap. III (p. 123) for complete discussion of crowns.

Superelevation on Curves.—Superelevation was discussed in Chap. III (p. 127).

Limitations Imposed by Steep Grades.—See page 425.

Specifications.—See Part III (pp. 1454 to 1455).

Construction Equipment.—Equipment is discussed in Chap. XV (pp. 1261 to 1262).

Inspection Details.—See Chap. XVI (pp. 1272 to 1296).

General Maintenance Methods.—Detail maintenance methods and costs are given in Chap. VII.

Well-defined holes are repaired by cutting out the edges square and vertical, filling with regular top stone, rolling, and puddling as in the original construction. Small depressions are usually remedied by filling with cold patch or screenings and oil. Surface pitting is remedied by oiling and cover. Shoulders are built up by scraping fresh dirt or by the addition of gravel or stone. The maintenance of ditches, guard rail, culverts, etc. is similar for all types of improvement.

Penetration Bituminous Macadam Pavements

General Discussion.—Bituminous macadam is what the name implies, a macadam pavement bound with bituminous material. The larger stone fragments, ranging in size from 1 1/4 to 2 1/2", depending on the depth of the course, are spread and rolled. A heavy grade of refined tar, residuum bituminous material, or fluxed natural asphalt is then poured hot, either by hand or machines, into the voids of the stone so that the stone fragments are covered with a thin coat of bituminous material; 3/4" stone or dustless screenings are spread over the surface, which is broomed and rolled until the voids are filled. If a bituminous flush coat is to be used, the excess filler is broomed off and the surface application of bitumen completed. Where the flush coat is not applied, a wearing coat of clean screenings is spread over the surface.

The amount of bituminous material used as binder varies from 1.50 to 1.75 gal. per square yard, depending on the depth of the course. The amount used for flush coats ranges from 0.2 to 0.5 gal. per square yard. Standard Specifications are given on page 1458.

Penetration-bituminous-macadam pavements are satisfactory and economical under normal traffic conditions ranging from 800 to 2000 vehicles daily (10-hr. count in summer season). They have been used quite extensively for traffic as low as 300 daily and in a few cases with moderate success up to 6000 daily, but caution should be exercised in their selection for roads carrying more than 2000 daily. The essential advantage of this types lies in its ability to provide very satisfactory road service at less initial cost than any type of rigid pavement. Under moderate traffic the final cost, including maintenance and renewal, compares favorably with any type of road. When well designed and constructed, with thickness recommended on page 391, these roads can be utilized in the future as bases for higher-type surfacings. They are a very satisfactory type for Class II traffic and for residential village and city streets. They are not seriously damaged by settlement and can be easily and quickly repaired. They are safe for high-speed travel; they will ride well if carefully constructed, but under careless inspection, where the bitumen is unequally applied, they gradually develop humps and hollows which are very disagreeable to fast traffic. This type of pavement requires careful construction and ample stone depth. They are resilient under steel-tire traffic as well as under ordinary motor traction, and for this reason are used extensively in England. They have a pleasing neutral color.

Bituminous macadams, including base and top, range in initial cost (1922 cost conditions) from $1.60 to $3.40 ($2.20 average) per square yard, depending on varying conditions of foundation soil, traffic, and local materials; the maintenance charge will range from 2 to 6 cts. per square yard per year, depending on all sorts of conditions, and the surface renewal charge distributed over the life of the pavement will probably average about 10 cts. for Class III traffic, 12 cts. for Class II traffic to 20 cts. for Class I traffic per square yard per year, 1926 cost conditions.[1]

The use of excellent materials is desirable, but for the type of traffic usually served best by this pavement (Class II) excessive refinements of materials or manipulation which raises the cost needlessly may well be avoided. Materials, details of design, and the essentials of inspection are later discussed. The following designs can be used with assurance of reasonable success.

Recommended Design.—The recommended designs represent conservative practice. A large mileage of thinner pavements has been constructed and has served quite satisfactorily for a considerable terms of years under favorable traffic conditions. As a matter of fact, it is rare that any well-constructed bituminous-macadam road of 80% of the thickness recommended is a serious failure, for if traffic proves too severe for either the foundation strength or surface resistance to wear, the old pavement can be strengthened by additional stone and the pavement made satisfactory by capping with a more resistant surface at considerably less cost than is required for a rigid type of pavement.

It is, however, undesirable to resort to additional surfacing in too short a period, particularly on the score on depth weakness, and it seems desirable to avoid much reduction in thickness below the depths recommended where local traffic laws permit a 28,000-lb. gross vehicle load limit. A surface life of 8 to 12 years is reasonable for this type when fairly well maintained under traffic of 800

[1] Renewal costs based on maximum allowable Vialog coefficient of 250 without excessive maintenance (see p. 551).

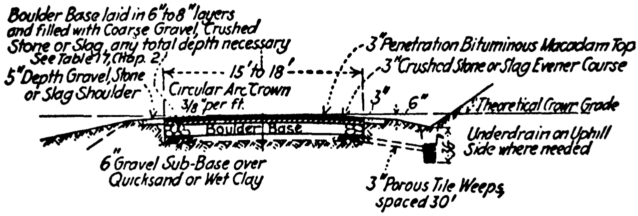


FIG. 152*A*.—Recommended penetration bituminous macadam pavement section, boulder base construction Class II or II*A* traffic. For discussion of crown see page 448.

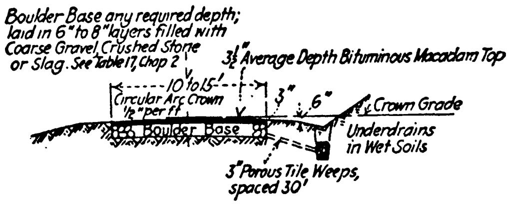


FIG. 152*B*.—Recommended penetration bituminous macadam pavement section, boulder base construction Class III or IV traffic.

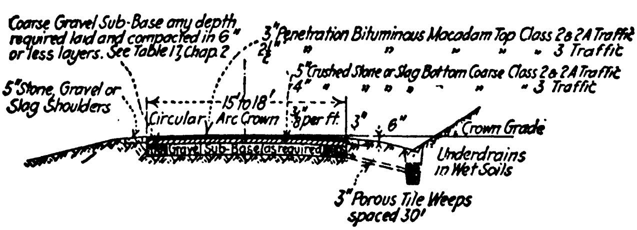


FIG. 153*A*.—Recommended penetration bituminous macadam pavement section, gravel base construction Class II, II*A* or III traffic.

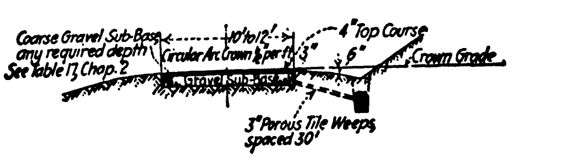


FIG. 153*B*.—Recommended penetration bituminous macadam pavement section, gravel, base construction Class IV traffic.

to 2000 daily, although a considerable number of cases do not fall within these limits (see Chap. VII).

Suitable Materials.—Materials for foundation courses were discussed under Water-bound Macadam (p. 441). Top-course materials are crushed stone and bitumen. Typical specifications are given on page 1383.

Crushed Stone.—Top-course crushed stone consists of three sizes, specified as follows:

Coarse aggregate.	1¼ to 2½" size
Intermediate size	¾ to 1¼" size
Dustless screening	⅛ to ¾" size

The stone must be hard, tough, and clean. The coarse aggregate 1¼ to 2½" size must be uniform and must not contain over 15% of stone smaller than 1¼" in size; this is important. Under heavy traffic where trap or the harder granites are available, a minimum French coefficient of hardness of 8 is desirable (5% or less of wear). Very good results can be obtained with a minimum hardness coefficient of 7 (6% or less of wear), and this is the usual minimum limit where limestones and the harder sandstones are the prevailing formation. Under light traffic and where hard rock is very expensive, a coefficient as low as 5 has been used, but under such conditions it is desirable to increase the size of the coarse aggregate to 2½ to 3½" and to use a harder grade of rock for the screening incorporated into the seal coat.

Bitumen.—Either asphalt or refined tars of the binder grade can be successfully used. Detail specifications are given on page 1388. For penetration bituminous macadam, an asphalt binder is preferable. The proper consistency of asphalt binder is about the only feature in design which varies with traffic and climatic conditions. The asphalt authorities believe that Table 89 is a safe guide for determining this requirement of asphalt binder. The author prefers 80–100 for medium-heavy traffic for western New York climatic conditions.

TABLE 89.—VARIATION IN PENETRATION OF BITUMEN

Traffic	Climatic temperatures		
	Low	Moderate	High
Light.	120–150	90–120	80–90
Medium.	90–120	90–120	80–90
Heavy.	80– 90	80– 90	80–90

Amounts of Material.—The amounts required for the foundation courses are given under Water-bound Macadam (p. 441).

For penetration top course, records in western New York show that the proportion of sizes used per cubic yard of finished top (limestone aggregate) are approximately as follows:

	Per Cent
1½ to 2½″ size crushed stone	65
¾ to 1½″ size	20
Dustless screenings	15

The total weight of all sizes of limestone, sp. gr. 2.7 (170 lb. per cubic foot solid), is about 4250 lb. per cubic yard for a 2½″ consolidated depth of finished macadam and about 4350 lb. per cubic yard for a 3″ consolidated depth.

The following table contains data released by the Asphalt Association:

ASPHALTIC MACADAM WEARING COURSE
(2½″ consolidated thickness)

Materials	Pounds per square yard
Coarse stone	227
Intermediate stone	45
Screenings	25
Asphalt	19

Bituminous Binder.—For single-coat work on steep grades, 1.5 gal. per square yard applied in one coat is satisfactory for 2½″ depth of top and 1.75 gal. per square yard for the 3″ depth. Where seal coats are used on the lighter grades, 0.4 to 0.5 gal. is generally used. Asphalt binder for penetration macadam weighs about as follows:

ASPHALT

Specific gravity	Weight, pounds per gallon
1.00	8.33
1.02	8.50
1.04	8.66
1.06	8.83

Design.—The principles of design are essentially the same as for water-bound macadam; that is, a good design considers the varying depth required for the prevailing soils, the most economical use of local materials, width, crown, and depth of top course. To determine total depths, reference is made to Table 74 (p. 391). Having decided on the depths required, alternate estimates are prepared on the basis of utilizing gravel, boulder, or macadam foundation courses. The variations in depth of macadam over gravel or boulder bases were developed on page 390. The most economical combination is adopted. The important point is that a real design shall be made, considering soil conditions and local material. Rigidly standardized uniform depth over varying soils is to be avoided (see sample design report, p. 1095). Where a macadam bottom or evener course is used, it is permissible to reduce the top to a 2½″ thickness under fairly light traffic. Under heavy traffic a depth of 3″ is recommended. Some states use a 2″ depth of top, but experience has not been entirely satisfactory with depths of less than 2½″ where a limestone aggregate is used.

Widths and Crown.—Widths were discussed in detail in Chap. III. Extra widening at sharp curves is desirable (see Tables 37*A* to 37*B*, p. 132). Village-street widths are outlined in Chap. III.

A circular arc crown is a good form. The total difference in elevation between center and edge of pavement is usually based on 5/16 to 3/8″ multiplied by the half the width in feet. For the usual road 16 to 20′ in width, the slope of 5/16″ per foot seems satisfactory. For village streets of greater width it is just as well to use the crown recommended for sheet-asphalt pavements as given on page 505.

On sharp curves on rural highways, one-way banking is desirable except on high fills. Ordinary practice in regard to banking slopes is given on page 127.

Steep Grades.—See pages 425 and 101 for limitations of use on steep grades.

Comparative Costs.—The cost of one-coat 2½″ bituminous top, using 1.5 gal. per square yard, will range from 80 cts. to $1, and a 3″ one-coat top, using 1.75 gal. per square yard, from $1 to $1.20 a square yard. The flush coat using 0.4 gal. per square yard will add about 8 cts. to the above costs. For comparison with macadam, a fair set of prices is (1922 cost conditions):

	Price per Square Yard
3″ water-bound macadam top course.....	$0.70
2½″ bituminous top, one coat of bitumen	0.85
2½″ bituminous top, flush coat..........	0.90
3″ bituminous top, one coat of bitumen...	1.00
3″ bituminous top, flush coat..........	1.10

Specifications.—See Part III, pp. 1458 to 1459.

Construction Equipment.—See Chap. XV (pp. 1262 to 1263).

Inspection Details.—See Chap. XVI (pp. 1296 to 1297).

General Maintenance Methods.—For detail methods and suitability of materials, see Chap. VII.

Well-defined holes are repaired by digging out the top course for the entire depth, out to the outer limits of the depression; squaring up the edges, and then filling in with regular top-size stone, 1¾ to 2½″; rolling or tamping; pouring with the same type of binder as the original course; adding ¾″ stone and screenings as in the original construction. Small holes are repaired by filling in with cold patch or screenings, and oil and surface pitting or livening of the bituminous binder is remedied by surface oiling, using an asphaltic oil if the binder is asphalt and a light tar if the binder is a refined tar. Shoulders, ditches, guard rail, etc. are routine procedure for all types of road as previously described.

RIGID-BASE PAVEMENTS

Cement-concrete Pavements

The usual concrete pavement is composed of a series of slabs separated by expansion and contraction joints. The concrete is generally the same mix for the entire depth, although some pavements have a special rich surface mix. Varying amounts of steel are used in the form of dowels, corner bars, tie bars, and mesh. Modifications of this type are constructed, such as Hassam concrete and concrete with thin bituminous surfaces. Specifications on page 1470 summarize the detail methods of construction. A large mileage of concrete pavement has been constructed on rural highways within the past few years and has met with popular approval.

From the engineering standpoint, concrete pavements are an excellent type for moderately heavy traffic (2000 to 5000 vehicles daily, 10-hr. count in the summer season). They have a gritty surface, which is safe for high-speed travel. They ride well if carefully constructed, particularly if built in separate strips 8 to 10′ wide, but under the usual inspection they are often rather bumpy and on grades of over 5% it is very difficult to get a smooth-riding pavement even with the greatest care. They have a bad glare in bright sunlight, but are easy to follow at night. For village or city streets, they do not look so well as pavements having a more neutral color, and in case they are used on curbed streets a high curb should be designed to permit future surfacing on top of the concrete. Concrete pavements cost from $2.50 to $4 per square yard ($3 average, 1926) for new construction; the maintenance charge will range from 0.3 to 3.0 cts. per square yard per year, depending on all sorts of conditions, and the surface renewal charge distributed over the life of the pavement will probably average about 10 to 14 cts. per square yard per year, 1926 cost conditions.[1] They are an excellent type for the lower-limit Class I roads or the higher-limit Class II roads where there is no large percentage of steel-tire traffic, but from an economic standpoint they should be used with caution for traffic of less than 1500 vehicles daily. They are more suitable for country highways than for village or city streets except alleys.

Concrete pavements give the advantages of the rigid-pavement type generally at less initial cost than other standard pavements, provided care is taken not to spend too much on refinements. When surface failure occurs they can be used effectively as a base for any of the standard paving surfaces. The essential advantage of the concrete pavement lies in its temporary use as a surfacing and its comparatively permanent value as a rigid base. To derive the most benefit from this combination of uses, it is necessary to

[1] Renewal costs based on maximum Vialog coefficient of 250 without excessive maintenance (see p. 551).

avoid excessive expenditure on refinements which have only a minor effect in increasing the life of the surface. A design which results in a surface life of 8 to 15 years, depending on traffic volume, seems rational for this type, considering the cost and the doubtful success of attempting to increase this period. This requires the use of excellent material and careful manipulation, but it does not require laboratory standards of mixing and curing nor does it require the extreme refinements of true reinforced-concrete design.

The cost of concrete pavements has been gradually increased by various refinements in materials, richness in mix, care in curing, joint details, reinforcement, etc. The object of all the refinements above 1 : 2½ . 5 concrete suitable for paving base of two-course pavements is to produce a wearing surface for traffic which will serve satisfactorily for a reasonable length of time before recapping with some standard form of renewal surface. Eventually, all concrete roads will probably serve as bases for asphaltic concretes, brick, asphalt block, stone block, or some form of repairable surface. It seems probable that the limit of justifiable expenditures for additional refinements in design and construction has about been reached, and in some cases exceeded. No matter what care is taken, spauling at joints, surface scale and pitting, vibratory disintegration, and weather action produce a rough unsightly surface in a comparatively short time, and it seems good policy to get merely a reasonably good concrete of adequate depth with the idea of resurfacing as it is needed. There is considerable disagreement concerning the practical limitations of materials, manipulation, etc. to carry out this general policy, but the following data can be used with assurance of reasonably good results.

Recommended Design (Concrete Pavements).—The recommended designs represent conservative practice. A large mileage of thinner roads has been built and has served quite satisfactorily for short periods of time. Any well-constructed concrete road of 6½″ average depth or more cannot be a serious failure, for if the traffic proves too severe a test for the concrete as constructed it can be protected and strengthened by capping with some standard form of surface which will make the road suitable for the heaviest modern loads.

It is, however, undesirable to resort to additional surfacing in too short a period, particularly on the score of depth weakness, and there seems to be considerable doubt as to the advisability of reducing strength below the limits shown in the following sections where local traffic laws permit a 28,000-lb. gross vehicle load.

These designs are based on the general line of reasoning outlined in the discussion following the sample designs and theory developed pages 392 to 427.

Recommended Design 1 (see Fig. 154). *Suitability.*—Rural highway traffic range of 4000 to 6000 vehicles daily (10-hr. count in summer season) in localities having a 28,000-lb. gross vehicle load limit.

Width.—Twenty feet constructed one-half at a time.

Crown.—Circular-arc radius 240′. Edge 2⅛″ below center line.

Concrete.—A 1 : 1½ : 3 mix with cement content not to exceed 1.9, and crushing strength, 28-day age, of from 3000 to 4000 lb. per square inch.

Transverse Joints.—Three-eighths-inch premolded asphalt spaced 33′, care being taken to insure vertical position of joint. Joints to be ½″ deeper than pavement and cut off after completion of pavement. Concrete to be rounded with an edge radius ¼″ on both sides of joint. Joints to be backed with 16-gage steel plates, ½″ less depth than pavement on grades of 4 % or more to insure vertical position. *No dowels or grooves at joints.*

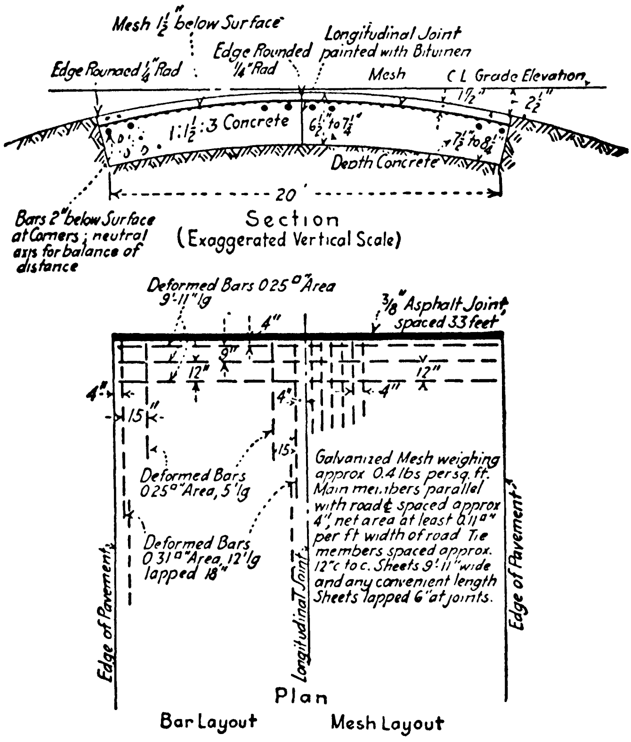


FIG. 154.—Recommended Design 1.

Depth of Concrete.—Based on Table 86 (p. 426).

	Outside Edge, Inches	Interior Areas, Inches
Ordinary soils	8¼	7¼
Gravel or macadam	7½	6½

Over freshly back-filled culvert trenches over 5′ wide increase depth and place bottom reinforcement to conform to culvert slab practice.

Reinforcement.—Combination of mesh and bar. Galvanized mesh to weigh not less than 0.4 lb. per square foot with the main members having an effective area of not less than 0.11 sq. in. per foot width of mesh and spaced not over 4″ C. to C., the tie members to be spaced not over 12″

C. to C. Size and spacing of corner bars based on formula $S = \frac{2.4W}{d^2}$, with $W = 14{,}000$ lb. (see p. 415). Percentage area of reinforcement of corner bars approximately three-tenths of 1 %. Tie bars, arbitrary assumption as shown.

QUANTITIES REQUIRED PER 100′ OF ROAD FOR THIS DESIGN

	New Grading	Resurfacing Jobs
Concrete, cubic yard	47.85	44.77
Mesh, square feet	2100	2100
Steel, pounds	860	860
Expansion joints, linear feet	60	60

APPROXIMATE COST (1922) PER SQUARE YARD

	New Grading	Resurfacing Jobs
Concrete	$2.70	$2.59
Mesh	0.28	0.28
Steel	0.10	0.10
Expansion joints	0.03	0.03
Total	$3.20	$3.00

A 9¼″ depth of plain concrete which is equal in strength to this design costs approximately $3.40 per square yard for new grading conditions.

Recommended Design 2 (see Fig. 155). *Suitability.*—For a rural highway traffic range of 1500 to 4000 vehicles daily (10-hr. count in summer season) in localities having a 28,000-lb. gross vehicle load limit.

Width.—Eighteen feet constructed one-half at a time.

Crown.—Circular-arc radius 200′. Edge 2″ below center line.

Concrete.—A 1:1½:3 mix with cement content not to exceed 1.9, and crushing strength, 28-day age, of from 3000 to 4000 lb. per square inch.

Transverse Joints.—Same as Design 1.

Depth of Concrete.—Based on Table 86 (p. 426).

	Outside Edge, Inches	Interior Areas, Inches
Ordinary soils	8½	7¾
Macadam or gravel	7¾	7

Over freshly back-filled culvert trenches over 5′ wide increase depth and use bottom reinforcement to conform to standard culvert slab practice.

Reinforcement.—Bars only. Size and spacing of corner bars based on formula $S = \frac{2.4W}{d^2}$, with $W = 14{,}000$ lb. Percentage area of reinforcement of corner bar approximately one-fourth of 1 %. Tie bars, arbitrary assumption as shown.

QUANTITIES PER 100′ OF ROAD FOR THIS DESIGN

	New Grading	Resurfacing Jobs
Concrete, cubic yards	44.44	41.97
Steel, pounds	1110	1110
Expansion joints, linear feet	54	54

APPROXIMATE COST (1922) PER SQUARE YARD

	New Grading	Resurfacing Jobs
Concrete	$2.83	$2.63
Steel	0.14	0.14
Expansion joints	0.03	0.03
Total	$3.00	$2.80

A plain concrete pavement of equal strength costs about $3.30 per square yard.

Recommended Design 3 (see Fig. 156). *Suitability.*—For rural highway traffic of less than 1500 daily. For these traffic conditions a concrete pavement is rarely justified from an engineering standpoint. Local demand, however, sometimes requires this type under these conditions, and for the purposes of expediency it is often desirable to meet the demand. Under

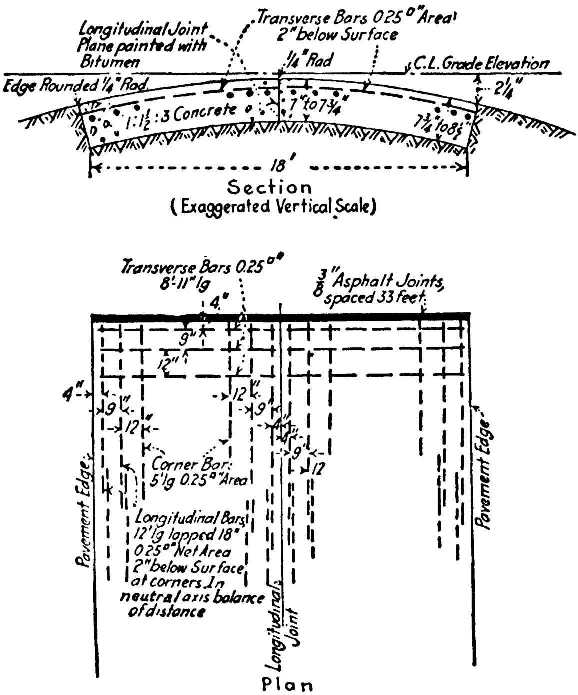


FIG. 155.—Recommended Design 2.

these conditions it is just as well to cut the first cost to a minimum and design the pavement with the idea of recapping with asphalt, etc., as soon as it shows the need. It should, however, be made strong enough so that an asphalt surface will raise its strength to that required for main-road loads.

Width.—Sixteen to eighteen feet recommended, constructed one-half at a time, or if full width is constructed at once, use submerged longitudinal steel joint.

Crown.—Circular-arc radius 200'. Edge 2" below center line.

Concrete.—A 1:2:4 mix with cement content of not over 1.7, and 28-day crushing strength of 2500 to 3500 lb. per square inch.

Transverse Joints.—Same as Design 1.

Average thickness of concrete 6½ to 7" for construction on new grading ordinary loams and clays, based on formula $d = \sqrt{\frac{1.5W}{S}}$, with $W = 12{,}000$

and $S = 360$ lb. For gravel soils or resurfacing on top of old macadam cut depth ½″ below formula.

Reinforcement.—Corner bars based on formula $S = \frac{2.4W}{d^2}$. Percentage area approximately four-tenths of 1%. Single side tie bar.

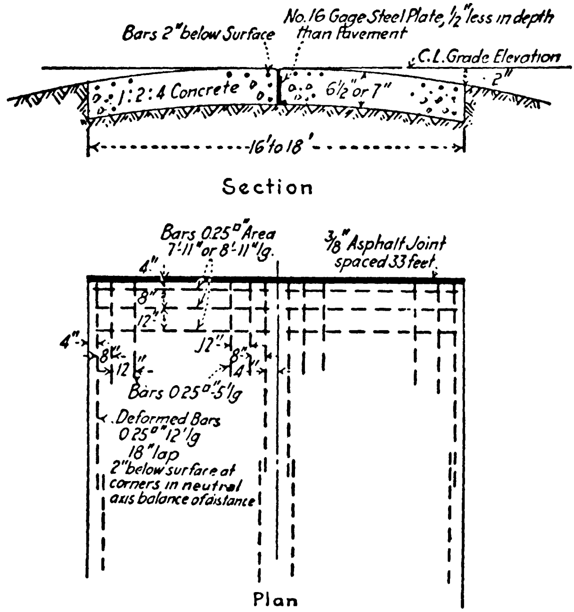


FIG. 156.—Recommended Design 3.

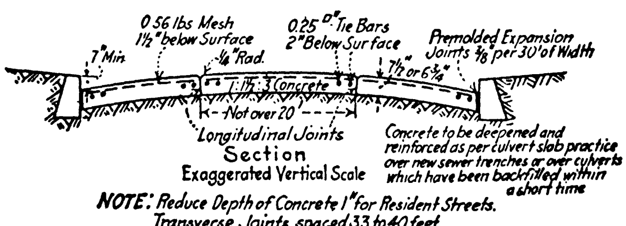


FIG. 157.—Recommended Design 4.

QUANTITIES PER 100′ OF 16′ ROAD FOR THIS DESIGN

	New Grading	Resurfacing Jobs
Concrete, cubic yards	34.6	32.1
Steel, pounds	820	820
Expansion, linear feet	48	48

APPROXIMATE COST (1922) PER SQUARE YARD

	New Grading	Resurfacing Jobs
Concrete	$2.24	$2.08
Steel	0.12	0.12
Expansion	0.02	0.02
Longitudinal steel joints	0.04	0.04
Total	$2.42	$2.26

Recommended Design 4 (see Fig. 157). *Suitability.*—For village business street, 28,000-lb. gross vehicle load limit.

Width.—Any desired total width constructed in longitudinal strips of from 10 to 20′ width. Longitudinal joints painted with bitumen. Expansion along gutter line and at intervals of 33 to 40′ across road.

Crown.—Circular arc producing depth of edge below center line as shown on charts (p 505).

Curb.—Seven inches high to permit future surfacing of asphaltic concrete or thin block.

Concrete.—A 1:1½:3 mix with cement content of not to exceed 2.0, and crushing strength, 28-day age, of 3000 to 4000 lb. per square inch.

Transverse Joints.—Three-eighths inch premolded asphalt spaced 33′. Special care to keep straight and at right angles to curbs.

Depth of Concrete.—Seven and one-half inches uniform on loams and clays. Six and three-fourths inches uniform on gravels and coarse sands.

Reinforcement.—Same as Design 1 except direction of main members of mesh, which are at right angles to the direction shown in Design 1, where longitudinal joints are more than 12′ apart.

Mesh.—To weigh not less than 0.56 lb. per square foot and to have an effective area of 0.14 sq. in. per foot width. Corner bars three-tenths of 1% of section area for 30″ back from corners.

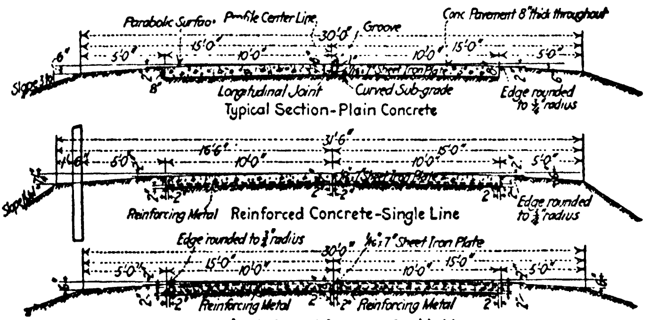


FIG. 158.—State of New Jersey (1922) Standard plain and reinforced concrete pavements.

Summary of Current Practice.—The tabulation (p. 462) gives a quick means of noting the essential features of current practice. This table was compiled by the Portland Cement Association in 1922. The typical sections following show standard sections of various design:

Materials.—The materials required are cement, gravel, stone or slag, sand or screenings, water, expansion joints, reinforcement, and joint plates.

Cement.—The requirements of the Portland cement have been quite well standardized and the required quality can be obtained without much difficulty. All cement should, however, be tested and approved before use.

Cement specifications are given on page 1387.

Cement tests are described in Chap. XI (p. 734).

Gravel and Broken-stone Aggregate.—The essential qualities of the coarse aggregate are that it be a clean, hard, well-graded size of crushed-rock or screened-gravel product.

Where trap rock and the harder granites are available, the minimum hardness requirement for crushed stone is generally placed at French coefficient of 8 (5% of wear). Where the limestones, hard sandstones, and granite hardheads are the most

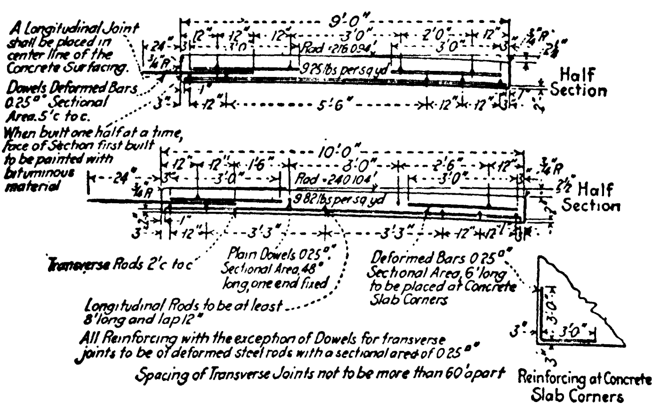


FIG. 159.—Massachusetts concrete road section (1922).

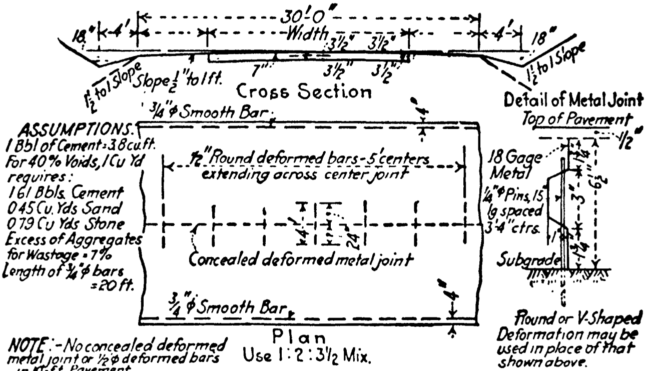
feasible source of supply, the minimum French coefficient can be dropped to 7 (6% of wear) with very good results. In exceptional cases a value as low as 6 (7% of wear) has been used with moderate success, but anything below 7 is risky for this type of pavement. The pavement is expensive and it is poor policy to use inferior materials which reduce the effectiveness of the result. The standard gravel abrasive test giving a 15% wear is about equivalent to a 5% wear of crushed-stone Deval test (see Chap. XI p. 742).

A graded size is desirable, but it is not feasible to pay too much attention to this factor. Current practice favors a maximum size of 2½ to 2¾" and a minimum of ½" for a small percentage of the product. The ½" size should not exceed 25% of the total coarse aggregate, which should be well graded between the ½ and 2¾" limits.

Typical aggregate specifications are given on pages 1383 and 1387.

Typical tests of stone are described in Chap. XI (p. 705).

Slag Aggregate.—Crushed blast-furnace slag has been given attention as a substitute for stone and gravel, as in some localities its use materially cheapens the pavement. There has been considerable doubt as to the effect of slag in combination with cement. The slag-concrete roads which have had service tests, in conjunction with the laboratory, extending over a considerable period indicate that a tough, strong concrete can be constructed of properly selected slag and that this concrete is not weakened by chemical



10-ft. Wide

	Cement bbls	Fine Aggr. Cu Yds	Coarse Aggr. Cu Yds	Concrete Cu Yds	Deformed Metal Joint lbs.	¼" Round Pins lbs.	¾" Round Smooth Bar lbs.	½" Deformed Bar, lbs.
100'	34.78	10 40	18 26	21 60	None	None	333.33	None
1 Mi	1836 38	549.12	964.13	1140 69	"	"	17599 82	"
1 Sq Yd	.313	.094	.164	.194	"	"	3.00	"

16-ft. Wide

100'	55.65	16 64	29 22	34 57	132.94	6 26	333 33	53.44
1 Mi	2938.32	878 59	1542 82	1825.30	7019 23	330 53	17 599.82	2821.63
1 Sq. Yd	.313	.094	.164	.194	.748	.035	1 875	.301

18-ft. Wide

100'	62 61	18 72	32 87	38 88	132.94	6 26	333 33	53.44
1 Mi.	3305 81	98842	1735 54	2052.86	7019 23	330 53	17599.82	2821.63
1 Sq. Yd	.313	.094	.164	.194	665	.031	1.667	.267

20-ft. Wide

100'	69.57	20.81	36 52	43.20	132.94	6 26	333.33	53.44
1 Mi	3673 30	1098.77	1928.26	2280.96	7019.23	330.53	17599.82	2821.63
1 Sq. Yd.	.313	.094	.164	.194	.598	.028	1.50	.240

FIG. 160.—Illinois concrete pavements (1922).

action due to the slag aggregate. Road concrete constructed of slag, however, tends to pit on the surface, due to a small percentage of pumice in the aggregate, and this is unslightly at best. Slag-concrete pavement serves the traffic quite satisfactorily, and in certain instances it can be used to advantage, but engineers, as a rule, prefer stone or hard gravel for most conditions of material

(text continued on page 460.)

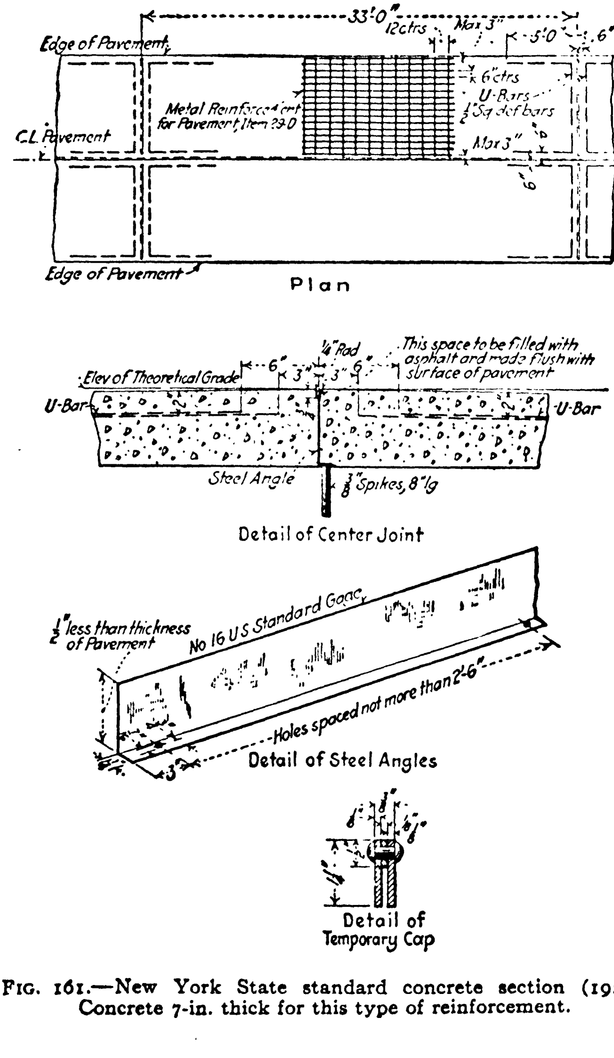


FIG. 161.—New York State standard concrete section (1922). Concrete 7-in. thick for this type of reinforcement.

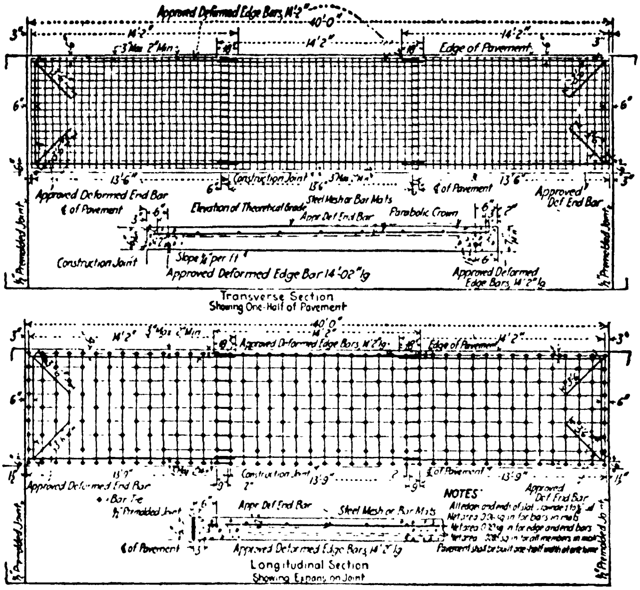


FIG. 162.—New York State (1923 standard) concrete pavement.

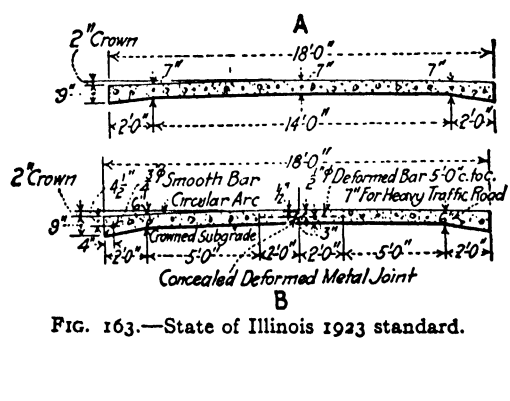


FIG. 163.—State of Illinois 1923 standard.

supply. Slag aggregate is entirely satisfactory for concrete-paving bases. Slag should weigh at least 1800 lb. per cubic yard and have a test of wear of not exceeding 12% (Standard Slag Abrasion Test).

Sand.—The sand used should be a hard, clean, well-graded sand free from organic impurities and containing only a small percentage of loam or silt. A hard, sharp, graded sand which does not pulverize under rolling pressure or water slack is essential.

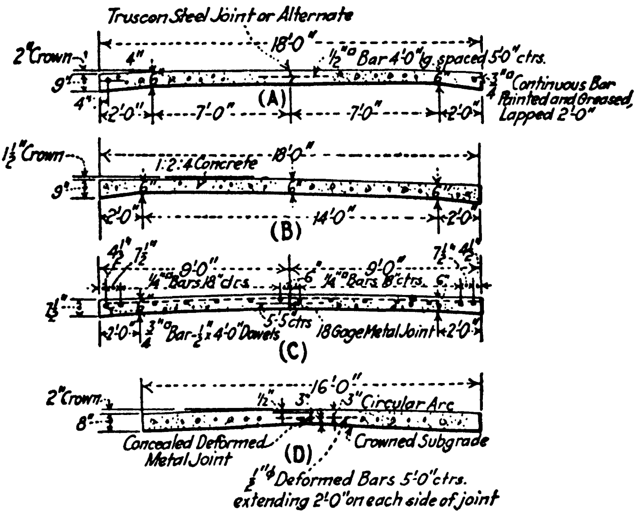


FIG. 164.—Typical state standards (1923), cement concrete roads

The sizing usually specified is as follows (percentages are by dry weight):

Per Cent

100 passing ¼" sieve.
20 minus passing No. 50 sieve.
6 minus passing No. 100 sieve.
5 minus loam and silt.

Sand and cement mixed in the same proportion required for pavement must develop at least as much strength as standard Ottawa sand[1] and cement mixed in the same proportions.

In exceptional cases, rock-crusher dust can be substituted for a part or whole of the natural-sand aggregate, but under the usual

[1] Ottawa sand is furnished by the Ottawa Silica Co. of Ottawa, Ill.; 100 % passes a No. 20 sieve and 100 % is retained on a No. 30 sieve. It has about 37 % of voids and a fineness modulus of 3.0.

conditions of material supply this expedient should be used with caution (see Specifications, and Sand Tests, Chap. XI).

Water.—Water shall be free from silt and organic matter and shall not show a strong alkaline or acid reaction (see Chap. XI for tests).

Expansion Joints.—The premolded type is the most practicable form for good work; the required properties and tests are given on page 1401.

Joint Plates.—Straight steel or corrugated steel plates Nos. 14 or 16 U. S. gage with flange on the bottom punched for spiking to the grade are as good a form as can be used. Sixteen gage is thin enough (see Figs. 161 and 163). It is believed that the straight plate is superior to the grooved or corrugated form, which tends to increase the liability of spalling.

Amounts of Material Required. *Amount of Water.*—The amount of water required will vary with the natural dampness of the loose aggregate, its sizing, and the cement content of the mix. For the usual 1 : 1½ : 3 concrete having a good workable consistency, the use of from 30 to 40 gal. per cubic yard is about right. Either excessively dry or sloppy concrete is undesirable. Some specifications describe slump tests to determine consistency (see p. 744). Such clauses are undoubtedly desirable in case of dispute, but for ordinary construction operations visual inspection is sufficient. The concrete should be plastic, that is, it should not be so wet that the mortar and stone separate when dumped or so wet that after rolling and finishing free water runs off the surface and drips off the forms. It is desirable for the mortar to flow slightly, as this produces a denser concrete with the usual amount of manipulation obtained under ordinary working conditions. Theoretically, fairly dry concrete is stronger than the plastic mix, but if a mix is so dry that it requires unusual care in spreading, tamping, and spading the actual result day after day is not so satisfactory as where the plastic mix is used.

Sprinkling subgrade takes from ½ to 1½ gal. per square yard for dry sand per square yard of pavement.

Curing concrete takes from 10 to 30 gal. per square yard if properly done, depending on weather conditions and time of year. (For the effect of curing on strength, see page 477.)

The total amount of water, all purposes, will range from 20 to 40 gal. per square yard, ordinary depths of concrete, or from 70 to 150 gal. per cubic yard of concrete.

Slump-test Values.—For defining the limits of allowable consistency in the Specifications, the slump-test definition is probably desirable. The standard test is described in Chap. XI (p. 744). According to Prof. Abrams of Lewis Institute, the following range will produce satisfactory consistency.

Machine finishing methods 1″ to 2″ slump.
Hand finishing methods 3″ to 4″ slump.

Amounts of Sand Stone and Cement.—Table 91 page 466 gives Abrams Quantity tables. Table 207 page 1092 gives other authority in amounts of aggegate. Fullers Rule is given on page 1092.

TABLE 90.—COMPARISON OF STATE HIGHWAY DEPARTMENT SPECIFICATIONS FOR CONCRETE FOUNDATIONS FOR PAVEMENTS WITH SPECIFICATIONS FOR CONCRETE PAVEMENTS COMPILED BY PORTLAND CEMENT ASSOCIATION
January, 1923

State	Date of specifications	Thickness of base	Proportions		Time of mixing in minutes		Fine aggregate					Coarse aggregate				
							Are specifications for base as rigid as for pavement?	Minimum percentage of strength of Ottawa sand in 1 : 3 mortar		Maximum allowable percentage of clay or loam in sand		Are specifications as rigid for base as for pavement?	Allowable per cent of wear for crushed stone		Maximum size in inches	
			Concrete base	Concrete pavement	Concrete base	Concrete pavement		Concrete base	Concrete pavement	Concrete base	Concrete pavement		Concrete base	Concrete pavement	Concrete base	Concrete pavement
Alabama	1920	..	1 : 9 1 : 3 : 6	1 : 2 : 3	1	1	No	75	100	5	3	No	...	5	3	2
Arizona	1922	..	1 : 3 : 6	1 : 2 : 3½	1	1	No	100	100	5	3	No	...	5	2½	2
Colorado	1922	..	1 : 3 : 6	1 : 2 : 3	1½	1½	Yes	100	100	3	3	No	5	4	2½	2
Delaware	1922	..	1 : 2½ : 5	1 : 2 : 4	1	1½	Yes	100	100	3	3	Yes	3⅓	3⅓	2¾	2¾
Florida	1919	..	1 : 3 : 6	Top 1 : 2 : 3	1	1	No	75	100	5	5	No	...	4	3	2
Georgia	1922	..	1 : 3 : 6	1 : 2 : 4	1	1	No	85	100	3	3	No	8	5	2¾	2¾
Illinois	1922	..	1 : 2 : 3½	1 : 2 : 3½	1	1	Yes	100	100	3	3	Yes	7	7	2½	2½
Indiana	1920	..	1 : 3 : 5	1 : 2 : 3	1	1	No	100	100	5	3	No	6	5	2½	2½
Kansas	1922	..	1 : 3 : 5	1 : 2 : 3½	1	1	No	80	100	5	3	No	...	7	3	2
Kentucky	1922	..	1 : 2½ : 5	1 : 2 : 3½	1¼	1¼	Yes	100	100	3	3	Yes	6	6	2½	2½
Maryland	1922	6	1 : 3 : 6	1 : 2 : 4	1	1	No	75	100	5	3	No	5	4	2½	2¾
Michigan	1922	..	1 : 2 : 4	1 : 1½ : 3	1	1	Yes	100	100	3	3	Yes	6	6	2½	2½
Missouri	1919	5	1 : 3 : 5	1 : 2 : 3	1	1	Yes	100	100	3	3	No	5	...	2	1½

Montana	1922	..	1:3:6	1:2:3	1	1	Yes	100	100	3	3	Yes	...	...	3	3
Nebraska	1922	..	1:3:6	1:2:3	1	1	Yes	100	100	3	3	No	6	4	2	2
New Jersey	1922	..	1:3:5	1:1½:3	1¼	1½	Yes	100	100	4	4	Yes	3½	3½	3	3
North Carolina	1922	..	1:2½:5	1:1½:3	1½	1½	Yes	100	100	3	3	No	5	4	2¾	2¾
Ohio	1922	..	1:2½:5	1:1½:3	1	1	No	85	100	5	3	No	8	6	2½	2½
Oklahoma	1921	5	1:3:6	1:2:3	1	1½	No	75	100	5	3	No	6	6	2	2
Oregon	1922	..	1:3:6	1:2:3	1	1	No	90	100	4	2	No	5	4	3	3
Pennsylvania	1922	..	1:2½:5	1:2:3	1¼	1¼	No	90	100	5	3	No	5	5	2¾	2¾
South Dakota	1920	..	1:3:6	1:2:3	1	1	No	75	100	5	3	No	...	5	3	2½
Utah	1922	..	1:3:6	1:2:3	1	1	No	90	100	5	3	Yes	5	5	3	2
Virginia	1920	..	1:3:6	1:2:4	1	1	No	85	100	5	3	No	8	5	2	2
West Virginia	1921	..	1:2½:5	1:2:2½	1	1	No	75	100	3	3	Yes	6	6	3	3
Wisconsin	1922	5	1:2½:5	1:2:4	1	1	Yes	100	100	3	3	Yes	6	6	2	2

NOTES.—Undated specifications listed with the date they were received from highway departments. Per cent wear listed is for crushed stone. Some specifications permit the use of pebbles or slag with lower per cent of wear, especially in the base. Specifications were not available for the states omitted.

Factors of Design.—The main factors of design are richness of mix, reinforcement, and joint details. Manipulation methods covering time of mix, care in curing, joints, etc., are discussed under Specifications page 1470, Inspection Details page 1299, to on page 1313.

Richness of Mix.—Richness of mix affects beam strength, resistance of the pavement surface to abrasion, and the imperviousness of the surface, seepage, and disintegration due to frost action. Road concretes in ordinary use (1926) range from 1:2:4 to 1:1½:3 mix with cement contents of from 1.6 to 2.1 bbl. per cubic yard of concrete.

While beam strength is slightly increased by added richness of mix, this factor has very little weight in the final decision, as there is no well-defined advantage of greater unit strength and less depth. The decision in regard to mix rests on its effect on abrasion and the porosity of the concrete, that is, richness above 1:2½:5 is entirely dependent on the production of a longer surface life before the pavement needs recapping. On this basis of reasoning, it is certainly not desirable to increase the cost over that required for 1:2½:5 mix by more than 10 to 15 cts. per square yard for each additional year of surface life. A mix of 1:2½:5 (1.25 bbl. of cement per cubic yard) taking traffic directly is not satisfactory or economical under moderate traffic, and even where such a surface is given a light protective coat of oil and screenings it rarely lasts over 3 to 5 years before a surface of asphaltic concrete or standard block is necessary. A considerable mileage of 1:2:4 concrete pavements (1.6 to 1.7 bbl. of cement per cubic yard) has demonstrated its ability to withstand moderate traffic up to 3000 rubber-tired vehicles daily (10-hr. count in summer) for a reasonable term of years (6 to 12) before recapping is desirable. The increased cost of this mix over and above the 1:2½:5 mix is most certainly justified as it amounts to approximately 0.1 bbl. per square yard or, in money, about 30 cts. per square yard initial cost, and probably adds at least 5 years to its surface life. In actual practice under the usual inspection and batching procedure, considerable variation of mix will occur, which produces occasional areas of concrete which are not true 1:2:4 mix. There have been enough failures of small areas, apparently due to porosity using this mix, to warrant a slight increase in richness to offset construction imperfections on even moderate-traffic roads, and on heavy-traffic roads a 1:1½:3 mix is apparently justified to increase density to withstand traffic pounding. The average life of such pavements cannot yet be set and will vary greatly due to different traffic and excellence of construction, but they seem to be about as expensive a mix as is desirable. A cement content of over 1.9 bbl. per cubic yard is of doubtful value and seems a waste of money except possibly on village pavements. This is not a needless caution, as the cement factor has been steadily rising, and while from the standpoint of the material industries it is desirable to sell all the cement possible, it is poor engineering to use a needless amount. The latest expedient is to call a sack of cement 0.95 cu. ft. in place of 1.0 cu. ft., thus raising the cement factor about 5%.

Tentative conclusions in regard to richness of mix may be expressed as follows:

For light- and moderate-traffic roads up to 3000 vehicles daily a mix of approximately 1 : 1¾ : 3½ with an average cement content of not to exceed 1.7 bbl. per cubic yard ought to give satisfactory results, provided the batching scheme prevents individual batches varying more than 7% from the average. (This is a reasonable limit of variation under modern specifications.) This mix should show a 28-day-age compressive strength of from 2500 to 3500 lb. per square inch.

For the heavier-traffic roads (3000 to 6000 vehicles daily) a mix of approximately 1 : 1½ : 3 with a cement content of not to exceed 1.9 ought to give satisfactory results and appears to be economically justified. This mix should show tests of 3000 to 4000 lb. at 28 days.

Table 91 page 466 shows Abrams' proportions for 3000-lb. concrete; it is an excellent general guide, but, considering the wide range in aggregate sizes as delivered, it is impracticable to cut the proportioning too fine. For explanation of fineness-modulus method of proportioning mix, see page 478.

Reinforcement and Joints.—The use of steel in concrete pavements is a comparatively recent development. Figures 154 to 164 show the different methods in use at present (1926). The wide variation in type, location, and function of the steel as used indicates conclusively that there is no certain knowledge as to the exact value of reinforcement in pavement design.

It is conceded that the use of some steel is practically and economically justified for equalizing construction imperfections in the concrete, for preventing interior cracks from spreading apart, for strengthening corners for cantilever load stresses and shear, and possibly for helping to keep adjacent slabs at the same elevation and to distribute wheel loads across joints. There is, however, considerable doubt as to the wisdom of the use of the amount of steel shown in some recent designs. Central areas of slabs are indeterminate in design, as there is no way of telling how they are stressed under the fluctuating conditions of traffic loads and subgrade support. It is uneconomical to attempt to increase the slab strength of central areas by the use of steel as a true reinforcement.

The design of depth for a pavement where reinforcement is used differs from that of plain concrete. The assumptions for the design of plain concrete slabs are quite rational and definite, namely, the greatest stress is produced by a live load at the extreme corner of a slab; if the slab is designed of uniform strength throughout and made strong enough to resist corner failure, the pavement should serve well, even though internal cracks occur, and the elimination of such cracks is not of much importance, except on the ground of appearance. These assumptions, however, require the use of a needless amount of concrete in internal areas where cracks do not develop. If internal cracks can be eliminated or reduced to an infrequent occurrence, it is evident that the use of steel at corners tends to produce a slab of more uniform resistance and makes it possible to reduce the required thickness of concrete. From an economic standpoint it is therefore desirable to use some steel for

(text continued on page 469.)

TABLE 91.—ABRAMS' TABLE OF PROPORTIONS AND QUANTITIES FOR 1 CU. YD. OF CONCRETE

(Based upon laboratory investigations, using approved materials, compressive strength, 28 days, with workable plasticity, 6 by 12" cylinders, 3000 lb. per square inch)

Size		Fine aggregates, screen per inch														
Coarse aggregate, inches	Cement in barrels aggregates in cubic yards	0–28			0–14			0–8			0–4			0–$\frac{3}{8}$"		
		Cement	Fine	Coarse	Cement	Fine	Coarse	Cement	Fine	Coarse	Cement	Fine	Coarse	Cement	Fine	Coarse
No. 4 screen to $\frac{3}{4}$	Proportions	1.0	1.3	2.4	1.0	1.6	2.4	1.0	1.8	2.3	1.0	2.0	2.3	1.0	2.7	1.5
	Quantities	1.96	0.37	0.69	1.85	0.44	0.66	1.82	0.48	0.62	1.75	0.52	0.59	1.79	0.72	0.40
No. 4 screen to 1	Proportions	1.0	1.3	2.7	1.0	1.6	2.6	1.0	1.8	2.6	1.0	2.0	2.5	1.0	2.6	1.8
	Quantities	1.90	0.36	0.76	1.77	0.42	0.68	1.72	0.46	0.66	1.67	0.50	0.62	1.72	0.66	0.46
No. 4 screen to $1\frac{1}{2}$	Proportions	1.0	1.2	3.1	1.0	1.6	3.2	1.0	1.7	3.1	1.0	2.0	3.0	1.0	2.4	2.4
	Quantities	1.82	0.32	0.84	1.68	0.40	0.79	1.63	0.41	0.75	0.61	0.47	0.72	1.62	0.57	0.57
No. 4 screen to 2	Proportions	1.0	1.2	3.5	1.0	1.5	3.5	1.0	1.6	3.7	1.0	1.9	3.6	1.0	2.2	3.1
	Quantities	1.75	0.31	0.90	1.63	0.36	0.85	1.55	0.36	0.85	1.52	0.43	0.81	1.53	0.50	0.70
No. 4 screen to $2\frac{1}{2}$	Proportions	1.0	1.1	3.8	1.0	1.4	3.9	1.0	1.6	4.0	1.0	1.8	4.0	1.0	2.1	3.5
	Quantities	1.72	0.28	0.97	1.58	0.33	0.91	1.51	0.35	0.89	1.49	0.40	0.88	1.50	0.46	0.78
No. 4 screen to 3	Proportions	1.0	1.1	3.9	1.0	1.4	4.1	1.0	1.5	4.1	1.0	1.7	4.1	1.0	2.0	3.7
	Quantities	1.69	0.28	0.97	1.58	0.33	0.97	1.49	0.33	0.90	1.49	0.37	0.90	1.49	0.44	0.81
$\frac{3}{8}$ to $\frac{3}{4}$	Proportions	1.0	1.3	2.3	1.0	1.7	2.3	1.0	1.9	2.3	1.0	2.2	2.2	1.0	2.8	1.4
	Quantities	1.96	0.37	0.67	1.85	0.46	0.63	1.82	0.51	0.62	1.75	0.57	0.57	1.79	0.75	0.37
$\frac{1}{2}$ to 1	Proportions	1.0	1.3	2.6	1.0	1.7	2.6	1.0	1.9	2.5	1.0	2.2	2.4	1.0	2.7	1.7
	Quantities	1.90	0.36	0.74	1.77	0.44	0.68	1.72	0.48	0.64	1.67	0.54	0.59	1.72	0.68	0.43

⅜ to 1½	Proportions	1.0	1.3	3.0	1.0	1.7	3.0	1.0	1.9	3.0	1.0	2.1	2.9	1.0	2.6	2.2
	Quantities	1.82	0.35	0.80	1.68	0.43	0.75	1.63	0.46	0.73	1.61	0.50	0.68	1.62	0.63	0.53
⅜ to 2	Proportions	1.0	1.3	3.3	1.0	1.7	3.4	1.0	1.8	3.5	1.0	2.0	3.4	1.0	2.4	2.9
	Quantities	1.75	0.34	0.86	1.63	0.41	0.83	1.55	0.42	0.80	1.52	0.45	0.77	1.53	0.62	0.66
⅜ to 2½	Proportions	1.0	1.3	3.7	1.0	1.6	3.7	1.0	1.7	3.9	1.0	2.0	3.8	1.0	2.3	3.3
	Quantities	1.72	0.33	0.95	1.58	0.37	0.87	1.51	0.37	0.87	1.49	0.44	0.84	0.50	1.51	0.74
⅜ to 3	Proportions	1.0	1.2	3.8	1.0	1.6	3.9	1.0	1.7	4.0	1.0	1.9	4.0	1.0	2.2	3.5
	Quantities	1.68	0.30	0.95	1.58	0.37	0.91	1.49	0.37	0.88	1.49	0.42	0.88	1.49	0.48	0.77
½ to ¾	Proportions	1.0	1.5	2.3	1.0	1.9	2.2	1.0	2.1	2.2	1.0	2.3	2.1	1.0	2.8	1.3
	Quantities	1.96	0.44	0.67	1.85	0.52	0.61	1.82	0.56	0.59	1.75	0.59	0.54	1.79	0.75	0.34
½ to 1	Proportions	1.0	1.5	2.5	1.0	1.9	2.5	1.0	2.1	2.4	1.0	2.3	2.4	1.0	2.8	1.6
	Quantities	1.90	0.42	0.70	1.77	0.50	0.66	1.72	0.53	0.61	1.67	0.57	0.59	1.72	0.72	0.41
½ to 1½	Proportions	1.0	1.4	2.8	1.0	1.9	2.9	1.0	2.1	2.9	1.0	2.2	2.8	1.0	2.7	2.1
	Quantities	1.82	0.37	0.75	1.68	0.47	0.73	1.63	0.51	0.69	1.61	0.52	0.66	1.62	0.65	0.51
½ to 2	Proportions	1.0	1.4	3.3	1.0	1.9	3.3	1.0	2.0	3.4	1.0	2.2	3.3	1.0	2.7	2.7
	Quantities	1.75	0.36	0.86	1.63	0.46	0.79	1.55	0.46	0.78	1.52	0.50	0.74	1.53	0.62	0.62
½ to 2½	Proportions	1.0	1.4	3.6	1.0	1.8	3.6	1.0	1.9	3.7	1.0	2.1	3.7	1.0	2.6	3.1
	Quantities	1.72	0.35	0.91	1.58	0.43	0.85	1.51	0.42	0.83	1.49	0.46	0.81	1.50	0.57	0.69
½ to 3	Proportions	1.0	1.3	3.7	1.0	1.8	3.8	1.0	1.8	3.9	1.0	2.1	4.	1.0	2.4	3.3
	Quantities	1.68	0.33	0.92	1.58	0.42	0.89	1.49	0.40	0.86	1.49	0.46	0.88	1.49	0.53	0.63
¾ to 1	Proportions	1.0	1.7	2.4	1.0	2.1	2.4	1.0	2.4	2.1	1.0	2.6	2.2	1.0	3.1	1.5
	Quantities	1.90	0.48	0.68	1.77	0.55	0.63	1.72	0.61	0.53	1.67	0.64	0.55	1.72	0.79	0.39
¾ to 1½	Proportions	1.0	1.7	2.7	1.0	2.0	2.8	1.0	2.3	2.7	1.0	2.5	2.7	1.0	3.0	2.0
	Quantities	1.82	0.46	0.73	1.79	0.50	0.70	1.63	0.55	0.65	1.61	0.59	0.64	1.62	0.73	0.48
¾ to 2	Proportions	1.0	1.7	3.1	1.0	2.0	3.1	1.0	2.3	3.1	1.0	2.5	3.0	1.0	3.0	2.4
	Quantities	1.75	0.44	0.80	1.63	0.48	0.75	1.55	0.53	0.72	1.52	0.56	0.67	1.53	0.68	0.55
¾ to 2½	Proportions	1.0	1.7	3.3	1.0	2.0	3.5	1.0	2.3	3.4	1.0	2.4	3.4	1.0	2.9	2.8
	Quantities	1.72	0.43	0.84	1.63	0.47	0.83	1.51	0.52	0.76	1.49	0.53	0.75	1.50	0.64	0.62

TABLE 91.—ABRAMS' TABLE OF PROPORTIONS AND QUANTITIES FOR 1 CU. YD. OF CONCRETE—(*Continued*)
(Based upon laboratory investigations, using approved materials, compressive strength, 28 days, with workable plasticity, 6 by 12" cylinders, 3000 lb. per square inch)

Size		Fine aggregates, screen per inch														
Coarse aggregate, inches	Cement in barrels aggregates in cubic yards	0–28			0–14			0–8			0–4			0–⅜"		
		Cement	Fine	Coarse	Cement	Fine	Coarse	Cement	Fine	Coarse	Cement	Fine	Coarse	Cement	Fine	Coarse
¼ to 3	Proportions	1.0	1.7	3.5	1.0	2.0	3.7	1.0	2.3	3.7	1.0	2.4	3.6	1.0	2.8	3.1
	Quantities	1.68	0.43	0.88	1.58	0.47	0.87	1.49	0.51	0.81	1.49	0.53	0.79	1.49	0.62	0.68
1 to 1½	Proportions	1.0	1.7	2.8	1.0	2.0	2.9	1.0	2.3	2.7	1.0	2.6	2.6	1.0	3.1	2.0
	Quantities	1.82	0.46	0.75	1.68	0.50	0.73	1.63	0.55	0.65	1.61	0.62	0.62	1.62	0.75	0.48
1 to 2	Proportions	1.0	1.5	3.2	1.0	1.9	3.5	1.0	2.2	3.3	1.0	2.4	3.3	1.0	3.0	2.6
	Quantities	1.75	0.39	0.83	1.63	0.46	0.85	1.58	0.51	0.76	1.52	0.54	0.74	1.53	0.68	0.59
1 to 2½	Proportions	1.0	1.4	3.4	1.0	1.9	3.8	1.0	2.0	3.7	1.0	2.3	3.7	1.0	2.7	3.1
	Quantities	1.72	0.35	0.86	1.58	0.45	0.89	1.51	0.44	0.83	1.49	0.51	0.81	1.50	0.59	0.69
1 to 3	Proportions	1.0	1.3	3.6	1.0	1.8	4.0	1.0	2.0	3.9	1.0	2.2	3.9	1.0	2.7	3.3
	Quantities	1.67	0.33	0.90	1.58	0.42	0.94	1.49	0.44	0.86	1.49	0.48	0.86	1.49	0.59	0.73

increasing corner strength. The rational and sure use of steel at corners with thinner concrete depends on the control of internal cracking.

The successful use of steel requires the subdivision of the pavement into slabs of such size that their integrity can usually be preserved under the action of traffic and frost. It is undesirable to reduce the size of slab below the maximum feasible size, as additional joints introduce additional corners, which are points of weakness, and more wear occurs at joints from spalling, etc. A small-sized slab also tends to rock under eccentric loading.

The decision in regard to spacing of contraction, expansion, and longitudinal joints should reduce internal cracking to negligible frequency. If transverse joints are provided at intervals of about 30 to 35′ and longitudinal joints are provided for each 8 to 10′ of pavement width, experience shows that controlled cracks are being provided for which correspond with the natural cracking tendency of concrete under frost and traffic action; that is, the adoption of this system of joints generally eliminates frequent internal cracks, provided the pavement has sufficient depth to handle traffic loads. This system of subdivided slabs with depths of 7 to 8″ of concrete will practically eliminate internal cracks, and if the corners are reinforced enough to prevent corner cracks for these depths, a fairly rational design results. This can be secured by corner reinforcement (see page 400). It is undesirable to reduce the thickness below the depths shown in Table 86. The proper use of corner reinforcement and tie bars around the slabs apparently justifies a reduction in the average thickness of the concrete of about 1 to 1½″ under that required for a plain concrete slab.

This saving justifies a maximum expenditure of about 4.5 cts. per square foot (1922 costs) for steel, or about 1.0 to 1.5 lb. of steel per square foot.[1] This amount is not often necessary or advisable. The use of approximately 0.5 to 0.8 lb. per square foot apparently gives a rational and economical result for most cases, and this amount is most certainly a good investment, considering pavement cost.

The use of steel in pavements may be briefly summarized as follows: Figures 154 to 157 show typical systems of reinforcement which appear logical.

Mesh and Bar Types of Reinforcement.—The mesh type having small unit members and close spacing is probably more effective than bars for equalizing construction imperfections, temperature, and moisture stresses. It can be placed closer to the surface of the pavement with less probability of chipping of the concrete than bar reinforcement. Theoretically, it has very little effect as a true tension reinforcement but its use seems beneficial in reducing longitudinal cracks in wide slabs. It is particularly useful on wide street pavements where it seems undesirable on account of appearance to divide the pavement into narrow strips.

[1] Mesh is an expensive form of reinforcement and should be used only on the heaviest type of pavement.

The bar type is better to hold adjacent slabs in contact after a crack occurs, and bars can be more scientifically arranged to give the most effective result where an effort is made to use steel as a true tension reinforcement. Bars, however, must be placed farther below the surface (at least 2″), and they are not very effective as a true tension reinforcement where the pavement depth is less than 7 to 8″. Bars can be most effectively used as a corner reinforcement and as ties around the outer edges of the slabs. Where used as ties, they are effective for any depth that may be adopted.

Where the bars are used, they are generally assembled into mats and tied together to insure proper spacing and elevation in the concrete. Theoretically, this may be desirable, but it requires additional isolated tie bars which have very little practical value. Personally, the author has never experienced any difficulty in placing bars by hand sufficiently close to their proper location, provided the reinforcement is all in the same horizontal plane. The man placing the steel must be intelligent, but his wage is more than offset by the saving in the labor of manufacturing the mats and in the saving of unnecessary tie bars.

The typical recommended sections (Figs. 154 to 157) assume that the bars are placed by hand and not wired. If it is desired to use the mat form, add tie bars at the longitudinal laps.

Corner Reinforcement.—Figure 135 (p. 401) shows graphically the effect of different percentages of steel on modulus of rupture; as discussed on page 398, it is probably not desirable to raise the design tension value above 65% of the modulus of rupture for 28-day concrete.

There are two formulas that can be considered in approximating the amount of reinforcement in corners. These, derived on page 415, are as follows:

$$d = \sqrt{\frac{2.6W}{S}} \qquad S = \frac{2.6W}{d^2} \qquad (1)$$

$$d = \sqrt{\frac{2W}{S}} \qquad S = \frac{2W}{d^2} \qquad (2)$$

Formula (1) is adopted for plain concrete and assumes approximately 85% of the wheel load carried by the corner. Formula (2) assumes approximately 66% of the wheel load carried by the corner. Formula (2) assumes the use of doweled or grooved joints to help transmit the load across the joint. Formula (1) is on the safe side, and as there is considerable doubt as to the practical effectiveness and desirability of doweled joints, it seems the more rational formula to use. For an 8″ depth of 1 : 1½ : 3 concrete pavement on a main road in states having a statutory limitation of 28,000-lb. gross vehicle load, the design wheel load plus impact is approximately 14,000 lb. and the value of S becomes

$$S = \frac{2.6(14{,}000)}{8^2} = 580 \text{ lb.}$$

A design tension value of 580 lb. requires at least one-fourth to three-tenths of 1% reinforcement. The corner reinforcement shown in Fig. 154 is based on this assumption.

In a similar manner Table 92 is constructed. It is intended merely as a rough guide, as the final decision rests on cut-and-try methods under actual traffic tests.

TABLE 92.—APPROXIMATE PERCENTAGE OF CORNER REINFORCEMENT

Depth of pavement, inches	Design wheel load W, in pounds, including impact allowance		
	8,000	14,000	20,000
6	0.3	Not suitable	Not suitable
7	0.0	0.4	Not suitable
8	0.0	0.25	0.5
9	0.0	0.0	0.35

TABLE 93.—AREA OF STEEL IN SQUARE INCHES PER FOOT WIDTH OF SLAB (CORNER REINFORCEMENT)

Depth of pavement, inches	Design wheel load W		
	8,000	14,000	20,000
6	0.22	....	....
7	....	0.34	....
8	....	0.24	0.48
9	....	....	0.38

Most corner cracks occur at a distance of less than 2½′ from the corner, so that if corner reinforcement with the effective area given in these tables is provided for a distance of 2.5′ back from the corner it is apparently a safe design. The recommended reinforcement layout of Fig. 154 is based on this assumption. A convenient size for corner reinforcement bars is ⅜ or ½″.

Side Bars.—The use of some kind of tie around the outer edges of the slabs is quite uniformly approved. Bars are effective in reducing spread of cracks which may develop. They increase the distribution of load across internal cracks by holding the parts in close contact and by dowel action. They should be strong enough to resist wheel-load shear after the crack forms. They have practically no effect in increasing strength or in the prevention of the formation of cracks. Convenient sizes in general use are ½ to ⅝″ bars (see Figs. 159, 160, and 164).

Dowels.—The advocates of the use of dowel-pin connections between adjacent slabs believe that such construction (see Fig. 159, p. 456) keeps adjacent slabs at the same elevation and reduces the load carried by the slab corner. The sizes and spacing in ordinary use are shown in Fig. 160; the dowels are coated with bitumen or wrapped in paper or encased in pipe to permit movement at transverse joints. There is comparatively little trouble in the matter of equal level of adjacent slabs where the joints are smooth and vertical. The few cases which occur due to defective construction can be easily remedied. Personally, the author does not favor the attempt to produce a 50% distribution across construction transverse joints by means of dowel connections.[1] It is doubtful if pin or groove joints can be constructed which permit contraction and expansion and yet are still very effective in permanently transmitting load. It is believed that it is a better general principle of design to make the corners self-supporting by means of corner reinforcement, the action of which is reasonably sure.

The recommended designs Figs. 154 to 157 are based on this principle of design.

Joint Details.—It is desirable to reduce the number of joints to a minimum, as they are generally the weak points in construction; they also tend to spall due to rub and traffic pounding. Also, enough joints must be used to permit a rational use of steel. Less spall, however, occurs at well-defined joints with rounded edges and a complete smooth separation of adjacent blocks than at natural contraction cracks or where the submerged type of joint is used.

The treatment of expansion and contraction is the source of very wide disagreement among engineers.

Expansion.—Some very excellent practice ignores expansion on the ground that the stress produced is well under the working stress of concrete in compression and that it is cheaper to repair infrequent blow-ups due to expansion than to spend the necessary money for expansion joints. (Expansion joints cost about 8 to 12 cts. per linear foot of joint and their use increases the pavement cost about 2½ to 4 cts. per square yard.) Personally, the author believes that all joints should be both expansion and contraction joints and that it is not wise to omit them.

Expansion produces a stress of approximately 13 lb. per degree of temperature where no movement occurs, and produces an elongation of free end slabs of approximately 0.008″ per 100′ of road per degree of temperature. For the extreme range of temperature in northern climates, expansion probably does not produce a compressive stress of over 1000 lb. per square inch where expansion joints are not used. This stress will not cause failure of the concrete but is sufficient to cause buckling or blow-ups under certain conditions. Expansion joints permit the free expansion of the individual slabs and prevent internal stresses. The space to be provided depends on the range of temperature expected; for northern climates with a maximum of perhaps 80° above the temperature of construction an allowance of approximately ¾″ per 100′ is necessary and 1″ per 100′ is not excessive. Where joints are placed every

[1] Tie bars at internal cracks are effective in producing distribution across such cracks, as no expansion or contraction need be provided at these points where the road is constructed with transverse joints every 33′.

33′, a joint thickness of 3/8″ is a good allowance; this corresponds with ordinary practice. Such joints are of the premolded type. It is essential that they be constructed in a vertical plane to prevent rising of slabs along the joint due to expansion pressure. It is also essential that a complete separation of slabs is obtained; that is, the bitumen must extend from bottom to top of the pavement and from edge to edge.

It is not difficult to keep these joints vertical on level grades, provided reasonable care is exercised as described in the specifications. On grades of 4% or more, however, it is advisable to back them up on the downhill side with 16-gage steel plates as shown in Fig. 161.

Transverse Contraction Joints.—While it seems advisable to make all transverse joints combined expansion and contraction joints spaced about 33′, there is considerable tendency to increase the distance between expansion joints to 100 to 300′ and to provide intermediate submerged contraction joints at intervals of 30 to 70′. Such joints are generally 14- or 16-gage steel plates cut one-half inch less depth than the pavement. The concrete is screeded and finished continuously over these joints and contraction produces a more or less jagged crack over the top of the plate which tends to spall more under traffic than the rounded-edge expansion joint; the argument for these joints is based on a smooth-riding pavement, but there seems to be no well-defined basis of fact in this contention; these steel plates cost about the same as a 3/8″ premolded joint.

Longitudinal Joints.—There are two general types: the submerged steel plate where the entire width of pavement is constructed at one time (see Fig. 161, p. 458), and the plane of separation joint where the pavement is constructed in successive strips. The second is the better method, as a better line and a denser edge are secured. With the submerged joint the concrete along the center steel is often porous, due to lack of care in manipulation; this condition cannot be observed and remedied as well as where the pavement is constructed in successive strips. In the second method a denser edge is insured, as when the forms are stripped it is easy to detect and correct carelessness in spading along the edge. It is important to get a denser impervious concrete along these joints, as water is bound to seep in and the concrete is subjected to its greatest strain at this location.

Design Details.—Design details consider depth, width, crown, and detail arrangement of joints and reinforcement.

Depth.—The design of depth was discussed on pages 392 to 428 summarized in Table 86 (p. 426).

Width.—Concrete pavements on rural roads range from 9 to 20′ in width. Single-track 9′ pavements are of doubtful value, for if a road has enough traffic to warrant the concrete type of pavement this certainly warrants greater width. These single-track concrete roads have not been at all satisfactory in localities with which the author is familiar. Widths of 18 or 20′ seem the most logical for conditions which make concrete suitable on rural highways. Extra width on curves was discussed in Chap. 3 and the results summarized on page 132. Current practice in this matter is indicated in Fig. 21 (p. 129). The widths of village streets are discussed on page 165. Alleys in business districts should be at least 18′ wide; in resident sections at least 16′.

Crown.—There are two popular shapes of crown curve, parabolic and circular. The parabolic is easier to figure, and for narrow rural pavements of 18′ or less in width which can be struck off by a single templet there is no objection to this type of curve. For wider pavements, particularly for street work, the circular arc is probably better, as a single templet can be used for the successive strips without danger of inequalities in the crown. Street *crowns* made up of a curved central portion and inclined-plane sides are to be avoided, as, while this is an easy form to construct, it does not look well, as it gives the appearance of a sag in the crown. The circular-arc crown is probably better adapted to all conditions.

The difference in elevation between the center line of the pavement and the edge of gutter line varies with width and grade. The adopted height of crown depends on convenience for traffic and appearance. On rural highways, for widths up to 20′, usual practice ranges between ⅛″ per foot of half width to ¼″ per foot. A total crown of 2″ for 16, 18, or 20′ total width of pavement serves very well. This requires a crown radius of from 200 to 250′.

On wider pavements and village streets the range in standard practice is shown in Figs. 167*A* to 167*B* (p. 505). Superelevation and shoulder and ditch grading were discussed in Chap. 3 page 124. Standard practice is indicated by Fig. 309 (p. 956).

In alley work a crown slope towards the center of ¼″ per foot is satisfactory. Alleys generally drain toward and along the center line with pick-up catch basins at short intervals.

Detail Arrangement of Reinforcement and Joints.—See Figs. 154 to 164 (pp. 451 to 460).

Steep Grades.—See pages 425 and 101 for limitations imposed by steep grades.

Specifications.—See Part III (p. 1470 to 1474).

Construction Equipment.—See (p. 1312 to 1263).

Inspection Details.—See Chap. XVI (p. 1299 to 1264).

General Maintenance Methods.—Detail maintenance is discussed in Chap. VII, (pp. 565 to 569). Concrete-pavement maintenance consists in the repair and sealing of cracks and joints, the replacement of disintegrated areas, and the renewal of shoulders. Cracks and joints are sealed with bitumen or a mixture of bitumen and sand. Disintegrated areas are either temporarily resurfaced with cold-patch bituminous mixtures or dug out and replaced with concrete, generally using a quick-setting cement (see Maintenance p. 565) to avoid long inconveniences to traffic. On lighter-traveled roads shoulders are renewed by scraping additional earth up along the edge of the pavement with a road scraper; on heavily traveled roads shoulders are built up solid along the edge by the use of gravel-oiled or cold-patch macadam.

RECENT CONCRETE DATA

At various places in the text reference is made to the effect of time of mixing on strength; care in curing on strength and fineness modulus on strength. The following data briefed from publications of the Portland Cement Association gives the most recent data (1923) based on experiments at Lewis Institute under the direction of Prof. Duff Abrams.

Very often both contractor and inspector are somewhat skeptical as to the necessity for the more recent specification clauses based on rational care in aggregate sizing, time of mixing and care in curing. Even a casual inspection of the experimental results here recorded ought to convince any unbiased inspector of the necessity of care in these particulars.

Proper Time of Mixing Increases Strength.—Effective mixing involves both time and number of revolutions. Many inspectors and contractors believe that the number of revolutions controls strength of mix and that by speeding up the mixer the time can be

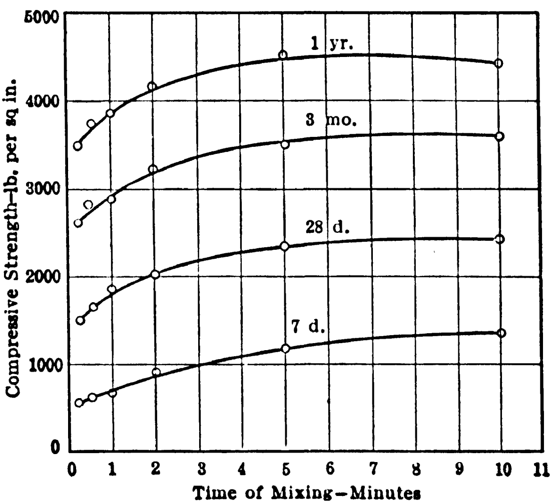


FIG. 1.—Relation between time of mixing and the compressive strength of concrete.

reduced without any detriment to the mix. Figures 1 and 2 show that the specification clauses involving both time and speed are essential clauses and that both must be complied with for the best results.

Proper Curing Increases Strength of Concrete.—Careful distinction should be made between the requirements of concrete for water during the mixing operation and in curing. A safe rule to follow is to use the smallest quantity of *mixing* water that will produce a sufficiently plastic mixture for the work in hand, and then to give the surface of the concrete as much *curing* water as possible after the concrete is placed.

Concrete hardens because of chemical reactions between Portland cement and water. Down to an amount less than can be used in construction work, the smaller the quantity of mixing water the stronger will be the concrete. Therefore, the quantity of mixing water should be reduced as far as possible. However, once the concrete is placed, and it has hardened, conditions change and ample curing water should be provided.

The chemical reactions of the hardening of concrete are slow, and if sufficient moisture is not present they cannot be completed. The mixing water essential to proper hardening of freshly placed concrete is often lost by absorption or evaporation even after the

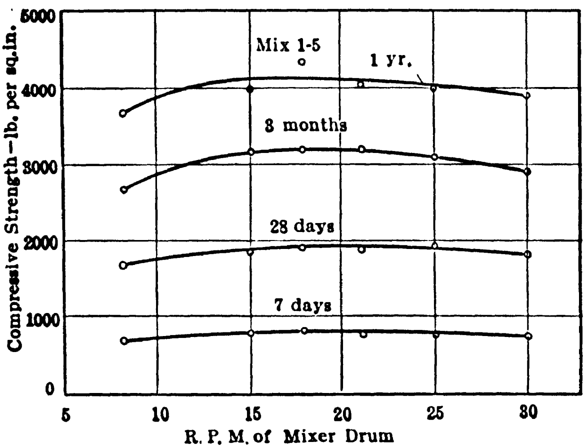


FIG. 2.—Relation between speed of mixer and the compressive strength of concrete

concrete has begun to harden. Under such conditions, concrete attains only part of its potential strength. Therefore, the water content of freshly placed concrete should be conserved. Keeping concrete damp during its early hardening period, or, in other words, providing plenty of curing water, prevents evaporation of necessary moisture, and permits concrete to harden under favorable conditions.

Protect Concrete While Hardening.—Tests show that protection during the early hardening period greatly increases the strength and resistance to wear of concrete. Figure 3 gives a summary of the results of these tests. All specimens were tested at the same age—4 months. One set was allowed to harden in air for the full 4 months the second set was stored in damp sand for 3 days and in air for the remaining 117 days; the third set was stored in damp sand 21 days and in air the remaining 99 days; while the fourth set was stored in damp sand the full 120 days and was tested while still damp.

Thus the increased strength and resistance to wear was caused solely by the better curing conditions provided.

Note that keeping concrete damp for the first 10 days increased its compressive strength 75%, for 3 weeks 115%, and for 4 months 145%.

Note also that keeping concrete damp for the first 10 days decreased the amount of wear 40%, and for 3 weeks 55%. Keeping concrete damp for 4 months did not cause a further decrease in wear, but the specimens were tested damp. Had they been allowed to dry out for a few days before being tested, the amount of wear would probably have been less.

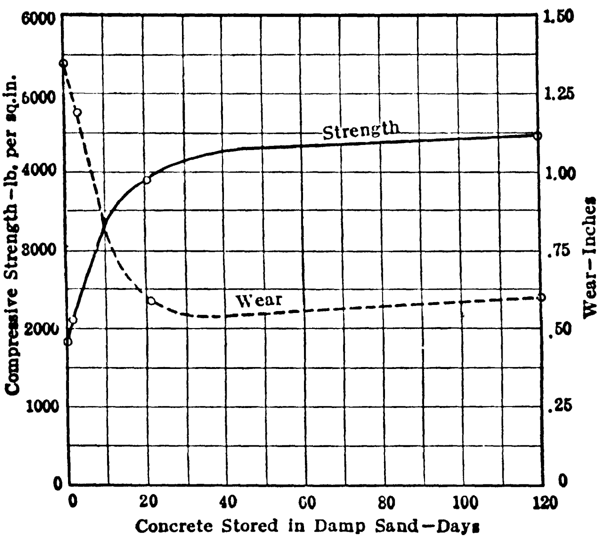


FIG. 3.—Keeping concrete damp the first ten days adds 75% to its compressive strength. It reduces the amount of wear 40% or *vice versa*, increases the resistance to wear 65%. Three week's protection adds still more strength and hardness.

Reversing the form of these wear results, 10 days protection increased the resistance to wear of concrete 65% and 3 weeks protection 120%. Thus proper curing increases the resistance to wear of concrete almost in the same proportion as the compressive strength. Needless to say, resistance to wear is an important consideration in floors, pavements and platforms.

Careful Grading of Aggregate Increases Strength of Concrete.[1] Portland cement, which forms only 15 to 25% of the volume of concrete, is carefully tested to see that it meets exacting specifications. Aggregates, which form 75 to 85% of the volume of

[1] Quoted from Pamphlet of Portland Cement Association.

concrete, should also be tested. Aggregates should be clean, free from coatings or organic impurities and structurally sound. Furthermore, proper size and grading of aggregates, or proper proportioning of fine and coarse aggregates, may double the strength of concrete, as shown in Fig. 4. Doubling the strength as the results of a few hours' study and a little closer supervision is surely worth while.

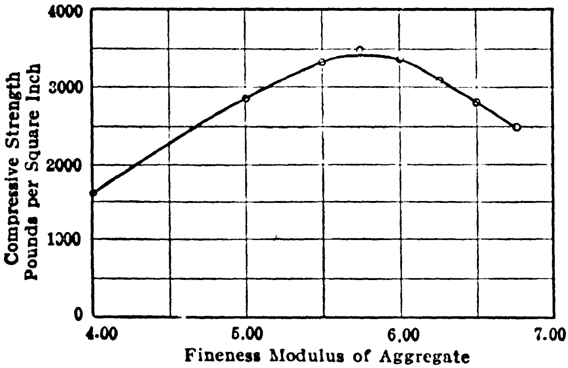


FIG. 4.—Relation between the grading af the aggregate and the strength of concrete.

Grading of Aggregates Varies.—Any number of fine and coarse aggregates of widely different size and gradings may meet the usual specifications in those respects. If combined in arbitrary proportions, such as 1:2:4, some of them will make far better concrete than others. For best results a proportion should be determined for each case that will fit the particular aggregates to be used. A little study will quickly show which of two or more available aggregates is the better graded and will therefore make the best concrete. Consideration of these factors may permit the use of local materials, with a saving in freight. It may also result in securing high-grade concrete at lower cost. Such studies can readily be made by any competent engineer or inspector, and at very small expense.

Assuming a given consistency, which should always be as stiff as the nature of the work permits, and a given mix (proportion of cement to total volume of mixed aggregate), the strength of the resulting concrete depends on the size and grading of the aggregate. In general, the larger and coarser the aggregate, the stronger will be the concrete. Coarse sand will produce stronger concrete than fine sand, while stone or pebbles in which the larger sizes predominate will produce stronger concrete than smaller stone or pebbles.

Fineness Modulus Shows Grading.—A simple index number, called the "fineness modulus," has been developed to indicate the size and grading of aggregates, and therefore their value for use in concrete. It can be applied to either fine or coarse aggregates, or to any combination of them. It shows at a glance which of two or more aggregates is the better graded. From it the proportions in which given fine and coarse aggregates should be combined are easily computed.

To determine the fineness modulus of an aggregate it is only necessary to make a simple sieve analysis. A set of U. S. standard square mesh sieves is used, each sieve having a clear opening double the width of the next smaller size. These sizes are 100, 50, 30, 16, 8 and 4 meshes per linear inch, and 3/8-in., 3/4-in. and 1½-in. For larger aggregates 3-in. and 6-in. sieves may also be used. The percentage of the aggregate, either by weight or by volume, coarser than each sieve is measured. The sum of these percentages, divided by 100, is called the fineness modulus. A close approximation can be obtained by using only alternate sieve sizes, 50, 16, 4 and 3/4, and estimating the percentages for the others. These four sieves make a convenient set for field use.

Table 2, on p. 479, gives the sieve analyses and fineness moduli of ten aggregates. The first is sand, the second, pebbles. The other

TABLE 2.—SIEVE ANALYSIS AND FINENESS MODULUS OF AGGREGATE

Ref. No.	Sieve analysis. Per cent coarser than each sieve									Fineness modulus	Comp. strength of a 1:4 mix. lb. per sq. in.
	100	50	30	16	8	4	3/8	3/4	1½		
1	99	93	63	40	20	0	Sand	..	..	3.15	
2	...	Pebbles		.	..	100	75	25	0	7.00	
3	99	93	68	55	39	23	17	6	0	4.00	1630
4	99	95	79	70	59	49	37	12	0	5.00	2910
5	100	95	84	77	70	62	46	16	0	5.50	3370
6	100	97	86	79	73	68	51	21	0	5.75	3540
7	100	98	90	85	79	74	56	18	0	6.00	3390
8	100	98	91	89	85	81	61	20	0	6.25	3150
9	100	99	94	93	90	87	65	22	0	6.50	2790
10	100	100	98	96	94	93	70	24	0	6.75	2540

eight aggregates were formed by combining these materials in various proportions. As the percentage of pebbles increases, the aggregate becomes coarser and the fineness modulus grows larger, thus reflecting the size and grading of the aggregate.

Good Grading Doubles the Strength.—The last column shows the compressive strength of concrete made from those eight aggregates in the proportions of one volume of cement to four volumes of total aggregate. These strengths are plotted in Fig. 4. Note that up to a certain point, the concrete increases in strength as the aggregate gets coarser. Beyond that point the aggregate is too coarse for the amount of cement used and the concrete decreases in strength. Note also that proper grading of aggregate may double the strength of the concrete. For other mixes the same principle holds true, although the greatest strength would be obtained with a different grading. The richer the mix, the coarser the aggregate and the larger the fineness modulus that will give the strongest concrete.

Aggregate No. 3, with a fineness modulus of 4.00, corresponds to a bank run gravel; it contains entirely too much fine material for best results. Aggregate No. 10, with a fineness modulus of 6.75, corresponds in grading to a crusher run stone; it contains too much coarse material. This shows the folly of using either bank run gravel or crusher run stone. Such material should always be screened and recombined in proper proportions.

Fineness Modulus is Simple.—The fineness modulus of any mix-of two or more aggregates, such as fine and coarse, is merely the weighted average of the fineness moduli of the separate materials. For example, if 32% of sand No. 1 ($F. M. = 3.15$) is mixed with 68% of pebbles No. 2 ($F. M. = 7.00$), the fineness modulus of the mixture is:

$$F. M. = \frac{32 \times 3{,}15 + 68 \times 7.00}{100} = 5.77$$

This agrees closely with the value given in the table (5.75) for aggregates No. 6, which was found by a sieve analysis.

Sieve analyses, which are easily made, will give the fineness moduli of any fine and coarse aggregates, while a simple calculation will show the fineness modulus of any mixture of them.

If the proper fineness modulus for the work in hand is known (page 482), the proportions in which given fine and coarse aggregates

TABLE 3.—STRENGTH OF CONCRETE FROM AGGREGATES OF UNIFORM FINENESS MODULUS

Mix 1:4 by volume. Relative consistency 1.10. Age at test 28 days. Specimens (6 by 12″ cylinders) were stored in damp sand and tested damp

Ref. No.	Sieve analysis. Per cent coarser than each sieve									Fineness modulus	Comp. strength lb. per sq. in.
	100	50	30	16	8	4	3/8	3/4	1 1/2		
11	95	89	82	75	67	67	67	62	0	6.04	2780
12	98	95	90	83	83	83	50	22	0	6.04	2530
13	98	94	90	86	83	80	55	18	0	6.04	2590
14	96	90	80	80	80	80	60	38	0	6.04	2570
15	90	85	81	78	75	73	66	56	0	6.04	2580
16	100	93	82	73	73	73	63	47	0	6.04	2880
17	100	100	100	92	81	60	45	26	0	6.04	2930
18	100	99	96	91	80	50	50	38	0	6.04	3040
19	99	98	90	85	80	76	38	38	0	6.04	3030
20	99	98	91	85	80	76	67	8	0	6.04	2690
Average											2760
Maximum value											3040
Minimum value											2530
Mean variation from average											6.2%

must be combined to secure that fineness modulus can readily be computed.

Sieve analyses of aggregates may vary over a wide range, but as long as the sum of the percentages coarser than each sieve (fineness modulus) is the same, the aggregates may be expected to give uniform results in concrete. This relation is shown in Table 3, which gives the sieve analyses, fineness moduli and concrete strengths of 10 aggregates made up by artificial grading of the same sand and pebbles. Although the sieve analyses vary, the total, or fineness modulus, is the same in all cases. Note that all 10 aggregates made concrete of substantially the same strength, the mean variation from the average being only 6.2%.

Thus the fineness modulus, which is an index of the size and grading, indicates the value of an aggregate for use in concrete. The reason is that, eliminating variations in the absorption of the aggregates, the fineness modulus reflects the quantity of mixing

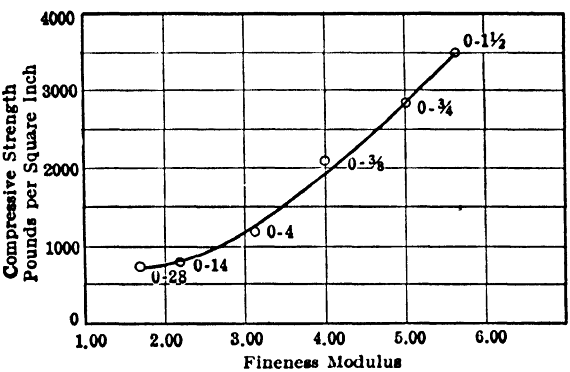


FIG. 5.—Relation between maximum size of aggregate, fineness modulus and strength of concrete.

water necessary to obtain a given consistency. In general, consistency and mix being the same, the fineness modulus indicates the strength that can be expected in the concrete. Other conditions being the same, aggregates having the same fineness modulus will produce concrete of substantially the same strength.

Proportions Depend on Aggregates.—Applying this principle to actual construction, it means that if several fine and coarse aggregates are available, concrete of substantially the same strength can be made from any combination of fine and coarse material, provided that the materials are proportioned to have the same fineness modulus. For one set of material the proper proportions may be 1 : 2.3 : 2.7, for another 1 : 1.6 : 3.4 and so on. To apply an arbitrary mixture, such as 1 : 2 : 3 to all aggregates, regardless of their grading, will not give uniform or satisfactory results. The mixture should be determined after a study of the particular aggregates to be used.

If the grading of the aggregate changes during the progress of the work it is an easy matter to change the mixture accordingly.

It is obvious that aggregates having a larger maximum size will have a larger fineness modulus. This is shown in Fig. 5. Note that as the maximum size increases, the fineness modulus increases, and also that the strength of concrete increases.

As stated before, larger fineness moduli can be used with rich mixes than with lean ones. Therefore, the best value of fineness modulus will depend on both the mix and the maximum size of the aggregate. Table 4, below, indicates the best value of the fineness modulus for the usual mixes and sizes of aggregates. The closer the aggregate comes to meeting these values the stronger will be the concrete.

These values for fineness modulus are based on sand and pebble aggregate for ordinary reinforced concrete. If crushed stone, slag or flat pebbles are used as coarse aggregate, or stone screenings as fine aggregate, reduce the values by 0.25. For mass work the values can be increased by 0.10 for 3/4″ aggregates, 0.20 for 1 1/2″ aggregate, and 0.30 for 3″ aggregate. *Fine aggregate used in concrete should not have a higher fineness modulus than that given for mortars of the same mix.*

TABLE 4.—BEST VALUES FOR FINENESS MODULUS

Mix cement—aggregate	Mortar	Size of aggregates					
	0–4	0–3/8 In	0–3/4 In	0–1 In.[1]	0–1 1/2 In.	0–2 In.	0–3 In.
1:7	3.20	3.95	4.75	5.15	5.55	5.95	6.40
1:6	3.30	4.05	4.85	5.25	5.65	6.05	6.50
1:5	3.45	4.20	5.00	5.40	5.80	6.20	6.60
1:4	3.60	4.40	5.20	5.60	6.00	6.40	6.85
1:3	3.90	4.70	5.50	5.90	6.30	6.70	7.15
1:2	4.20	5.05	5.90	6.30	6.70	7.10	7.55
1:1	4.75	5.60	6.50	6.90	7.35	7.75	8.20

[1] Considered as "half size" sieves; not used in computing fineness modulus.

Shrinkage of Aggregates after Mixing.—Table 4 gives the true mix, *viz.*, the proportion between the volume of cement (expressed as unity) and the volume of total aggregate after the fine and coarse have been combined. In the nominal mix, which is the one ordinarily used, the volumes of fine and coarse aggregates are given separately, *i. e.*, 1:2:4 or 1:1/2:3. The nominal mix can readily be converted into the true mix. *Vice versa*, if the ratio between fine and coarse aggregate is known, the true mix can be quickly converted into the nominal mix. For the usual ratios of fine and coarse aggregate (fine aggregate being 20 to 70% of the total) the volume of total aggregate after mixing will be about seven-eighths of the sum of the volumes of fine and coarse aggregate measured separately. For example, aggregate No. 6 in Table 2 contained 32% sand. Four cubic feet of the total aggregate would require 4.57 cu. ft. (4.00 × 8/7) of fine and coarse aggregate measured separately. Of that 4.57 cu. ft., 32% or 1.46 cu. ft. would be sand and 68% or 3.11 cu. ft. would be pebbles. A 1 : 4 true mix would be obtained by using 1 : 1.46 : 3.11 nominal mix. Because of this shrinkage when fine and coarse aggregate are combined a 1 : 2 : 4 nominal mix does not correspond to a 1 : 6 true mix as sometimes assumed, but to about 1 : 5.2 true mix.

More accurate values for the shrinkage of mixed materials can be obtained by measuring the volume of mixed aggregate obtained by mixing fine and coarse aggregates in the proper proportions, or by calculation from the weights of unit volumes of fine, coarse and mixed aggregates. For most cases, however, the average shrinkage of one-eighth is sufficiently reliable.

Example 1.—Four fine aggregates and three coarse aggregates are available for a certain job. Which should be used? Their sieve analyses give the following data:

Aggregate	Per cent coarser than each sieve									Fineness modulus	Range in size, in.
	100	50	30	16	8	4	3/8	3/4	1½		
Sand . . No. 1	100	90	70	55	35	20	0	0	0	3.70	0–3/8
Sand . . . . . No. 2	100	85	65	40	20	0	0	0	0	3.10	0–4
Sand . . . No. 3	95	75	60	30	0	0	0	0	0	2.60	0–8
Screenings . No. 4	85	80	75	35	25	0	0	0	0	3.00	0–4
Stone. . . . . . No. 5	100	100	100	100	100	100	100	40	0	7.40	3/8–1½
Pebbles. . . . No. 6	100	100	100	100	100	100	70	30	0	7.00	4–1½
Pebbles. . . . No. 7	100	100	100	100	100	100	45	15	0	6.60	4–1

Because of better size and grading (larger fineness moduli) sand No. 1 and stone No. 5 are the best materials. Sand No. 2 and pebbles No. 6 are good materials, but sand No. 3 is quite fine and pebbles No. 7 are rather small. Screenings No. 4 are well graded, but an equally well graded sand would be preferable, because rounded particles find their way into place with less mixing water. That is the reason why a higher fineness modulus is permissible when sand is used than when screenings are used (see text preceding Table 4). Therefore the other materials would be better. Any of these materials will make good concrete, however, provided the fine and coarse materials are proportioned to secure a combined fineness modulus suitable to the mix and maximum size of the aggregate.

Example 2.—A 1 : 4 true mix and a 1½″ maximum size of aggregate have been adopted. Sand No. 2 and pebbles No. 6 have been selected because their cost is less than that of the other materials. In what proportions should they be combined to get the best results? Or, in other words, what nominal mix should be used?

The proper value of the fineness modulus for these conditions is 6.00 (see Table 4). Then sand No. 2 and pebbles No. 6 must be combined in such proportions as to secure a fineness modulus of

6.00 for the total aggregate. Let P equal the percentage of sand and $100 - P$ the percentage of pebbles.

$$\begin{aligned} P \times 3.10 + (1.00 - P) \times 7.00 &= 6.00 \\ 3.10P + 7.00 - 7.00P &= 6.00 \\ 3.9P &= 1.00 \\ P &= 0.26 \text{ or } 26\% \end{aligned}$$

Thus of the total volume of aggregate, 26% should be sand and 74% pebbles. The true mix being 1:4 the sum of the volumes of sand and pebbles for a one bag batch measured separately will be 4.57 cu. ft. $(4 \times \frac{8}{7})$. Of this, 26% or 1.19 cu. ft. will be sand and 74% or 3.38 cu. ft. will be pebbles. The nominal mix will then be 1:1.2:3.4.

Example 3.—During the progress of the work it becomes necessary to substitute screenings No. 4 and stone No. 5. What change should be made in the nominal mix?

When screenings are used as fine aggregates or when crushed stone is used as coarse, the values for the total fineness modulus given in Table 4 should be reduced by 0.25. In this case screenings and crushed stone are both being used and the value of 6.00 for the fineness modulus used in Example 2 should therefore be reduced by 0.50, or to 5.50. Using the same notation as before:

$$\begin{aligned} P \times 3.00 + (1.00 - P) \times 7.40 &= 5.50 \\ 3.0P + 7.4 - 7.4P &= 5.50 \\ 4.4P &= 1.9 \\ P = 0.43 &= 43\% \end{aligned}$$

Dividing the total volume of aggregate before mixing (4.57 cu. ft.), 43% fine and 57% coarse, the new proportions are found to be 1:2.0:2.6.

AUTHOR'S NOTE.—The student should bear in mind that while the fineness modulus method is a valuable laboratory method for preliminary investigations it is not as practicable a method for specifying construction practice as the nominal mix regulated by maximum and minimum sizes and proportions of the sand and coarse aggregates.

The usual specification is based on proportions of mortar and coarse aggregate which give good practical results and the strength of these mixes as affected by fineness modulus is controlled by stipulating maximum and minimum percentages passing certain screens. See specifications page 1386 for sands; page 1384 for coarse aggregates.

INCREASED STRENGTH FROM RICHER MIXES

Tests clearly bring out the increase in strength resulting from the use of richer mixes. Typical results are given in the following table. The same aggregate, with a fineness modulus of 5.75 and graded up to 1½″ in size, was used in all tests. The cement was a mixture of equal parts of four brands purchased on the Chicago market. The relative consistency of all specimens was the same, 1.10, with a slump of 3 to 4″. This consistency is suitable for most construction work. All tests were made at the age of 28 days. Thus the increase in strength was due solely to increases in the quantity of cement.

Ordinary concrete mixes (1 : 3½ : 7 to 1 : 1½ : 2) contain from 3 to 8 sacks of cement per cubic yard of concrete. Note that within this range the strength of concrete increases in direct proportion to the quantity of cement. Each additional sack of cement per cubic yard adds about 500 pounds per square inch to the strength of concrete.

Effect of Quantity of Cement on the Strength of Concrete

True mix[1]	Approximate nominal mix[1]	Sacks of cement per cubic yard of concrete	Compressive strength, lbs. per square inch
Neat	....	29.7[2]	6630
1 : ⅓	....	23.0	5720
1 : 1	1 : ⅜ : ¾	15.7	5070
1 : 2	1 : ¾ : 1½	10.5	4070
1 : 3	1 : 1½ : 2	7.9	3570
1 : 4	1 : 1½ : 3	6.3	2760
1 : 5	1 : 2 : 3½	5.2	2090
1 : 7	1 : 3 : 5	3.9	1400
1 : 9	1 : 3½ : 7	3.1	1030
1 : 15	1 : 6 : 12	1.8	440

[1] The true mix is the ratio between the volume of cement, expressed as unity, and the volume of total aggregate after the fine and coarse have been combined. In the nominal mix, which is the one ordinarily used, the volumes of fine and coarse aggregate are expressed separately. The nominal mix can readily be converted into the true mix because for the usual proportions of fine and coarse aggregate (fine aggregate 20 to 70 % of the total) the volume of combined aggregate after mixing is about seven-eighths of the sum of the volumes of fine and coarse aggregate measured separately. Because of this shrinkage a 1:3:5 nominal mix does not give a 1:8 true mix as sometimes assumed, but about a 1:7 true mix.

[2] The volume of concrete obtained from neat cement depends on the quantity of mixing water used. For water-cement ratios between 0.3 and 0.6. the volume of concrete obtained from one sack (one cubic foot) of dry cement equals 0.5 of a cubic foot plus the amount of water used. In the example given here the water-cement ratio was 0.4 and the shrinkage was therefore about 10 %.

QUICK-HARDENING CONCRETE[1]

"Concrete work is usually allowed to stand many days before it is used. Quite often, however, *time* is the important consideration with new or repair work. The owner wanting early use of his improvement is not willing to wait the usual length of time and it is not necessary for him to do so. He can, with Universal Portland cement, secure in 3 days, concrete as strong as that ordinarily obtained in 28 days.

"High-early-strength concrete is particularly desirable on many jobs, such as special foundations and repair work that must be placed in use within a few days, sidewalks and street work where traffic demands that the improvement be placed in service at the earliest possible time, and frequently in

[1] Quoted from a pamphlet of the Universal Cement Co.

winter where it is desirable to get sufficient strength *quickly* to resist damage by freezing.

"**Simple Methods Accomplish the Result.**—To obtain this high-strength concrete in a few days, all that is necessary is to proportion, mix, place, and protect the concrete in the manner that actual construction experience and thousands of laboratory tests extending over a period of more than 5 years have demonstrated give the desired results. For example, a concrete pavement was built with Universal cement at the Chicago (Buffington, Ind.) plant of the Universal Portland Cement Co. which was opened to traffic when only 2 days old. This pavement has been carrying the heaviest loaded cement trucks for months without any damage to the concrete.[1]

"Simply *decreasing* the amount of water used in mixing greatly increases the 3-day strength of the concrete. Other factors also contribute toward giving a 3-day strength that exceeds the ordinary 28-day strength. The simple factors to obtain this quick-hardening, strong concrete are:

"1. Decreasing the amount of mixing water.
"2. Increasing the mixing time up to 5 min.
"3. Increasing the amount of cement.
"4. Placing the concrete at a temperature of at least 70°F.
"5. Keeping concrete at a temperature of 70°F. for 3 days.
"6. Keeping concrete damp for 3 days.
"7. Using calcium chloride where tests show it increases the strength.

"A good concrete such as is commonly used in building construction work is proportioned 1 sack of cement to 2½ cu. ft. of good sand and 4 cu. ft. of crushed stone or gravel, graded in size from ¼ up to 1½" material. If to the dry materials 7.7 gal. of water are added and the mixing time is 1 min., a concrete will be produced with the strength at different ages, shown in line A of the following table:

	Mix	Gallons water per sack cement	Minutes mixing time	Compressive strengths, pounds per square inch			
				1 day	3 days	7 days	28 days
A	1:2½:4	7.7	1	240	750	1320	**2600**
B	1:2½:4	7.7	5	340	910	1550	3030
C	1:2½:4	6.1	1	520	1350	2090	3700
D	1:1½:2½	5.5	1	560	1580	2530	4230
E	1:1½:2½	4.4	1	880	2110	3630	5250
F	1:1½:2½	4.4	5	1150	**2860**	4020	5740
G	1:1½:2½	5.5 (2 % $CaCl_2$)	1	1110	2090	3000	4170
H	1:1½:2½	3.4 (2 % $CaCl_2$)	5	1910	3380	4200	5260
[a]	1:1¼:2½	4.6	5	1580[a]			

[a] Concrete used at Chicago plant of Universal Portland Cement Co.

"**Quadrupling the 3-day Strength.**—Note that concrete A has 240-lb. compressive strength per square inch at 1 day, 750 lb. at 3 days, 1320 lb. at 7 days, and 2600 lb. at 28 days. But *quick-hardening* concrete with *higher strength* than concrete A in 3 *days* is wanted. Concrete B in the table is the same in every respect as concrete A, except that the mixing time was 5 min. instead of 1 min. This *one* change produced an increase in 1-day strength of 100 lb., or 42 %. By reducing the volume of the mixing water from 7.7 to 6.1 gal. per sack of cement, concrete C is obtained with 1-min. mixing, and with 5-min. mixing the concrete would be still better and stronger. The effect of increasing the amount of cement is shown by concrete D in the table. This is a 1: 1½: 2½ mix instead of a 1: 2½: 4 mix as in concrete A The workability or consistency of concrete D is exactly the same as is A, the mixing time is the same, but D has an increased amount of cement—about 2.1 bbl. per cubic yard as compared with about 1.4 bbl. for A. There is, however, a great increase in the strength at all periods. Decrease the volume of mixing water, making a somewhat drier mixture, and concrete E is the result. Mixtures rich in cement and with the minimum amount of mixing

[1] See table on p. 486 for strength of this concrete.

water require a maximum amount of mixing. Increase the mixing time for concrete E to 5 instead of 1 min., and strengths as in concrete F are obtained.

"Compare strengths at different periods of concretes F and A. Note that by simply increasing mixing time, decreasing volume of mixing water, and increasing quantity of cement, concrete F is obtained with a strength at 3 *days* that is *higher* than concrete A at 28 *days*.

"**Use of Calcium Chloride.**—Still higher-strength concrete at 1 and 3 days, such as concrete G, can be made by adding 2 lb. of calcium chloride per sack of cement. By further reducing the volume of mixing water, increasing the mixing time, and adding calcium chloride, concrete H with the strengths shown is obtained.

"With some cements little or nothing is gained in the way of increased 1- and 3-day strength by adding calcium chloride, while with some there is a marked increase in strength, as shown in the table. Tests should be made with particular materials to determine the effect of using calcium chloride in the mix. If the tests show that calcium chloride produces the desired increase in strength in 1 to 3 days, use it; if not, do not use it.

"The best way to use calcium chloride in the field is to add 100 lb. of commercial calcium chloride to 50 gal. of water in a barrel, stir until the calcium chloride is thoroughly dissolved and a standard solution with 2 lb. of calcium chloride to a gallon of water is obtained. Replace 1 gal. of mixing water *per sack of cement* in a batch of concrete with this standard solution and the right amount of calcium chloride is used.

"The results shown in the table are for concretes placed and maintained at a temperature of at least 70°F. for the time indicated. Increasing the temperature of the mix and the temperature during the curing of the concrete, and at the same time keeping the concrete damp, will give still greater strengths than are shown in the table. The use of steam at a few pounds' boiler pressure for curing will also greatly accelerate the rate of hardening of concrete and increase the 1- and 3-day strengths. Steam is available and may be used to advantage in many places and on many jobs.

"**Advantage in Cold Weather.**—High-early-strength concrete is particularly desirable during the fall and winter months, as such concrete has sufficient strength to resist being damaged by freezing much earlier than concrete proportioned, mixed, placed, and handled in the ordinary manner. This high-early-strength concrete also requires protection from freezing for a shorter period, which is desirable from the cost standpoint.

"**Strong Concrete in 3 Days.**—This quick-hardening concrete giving high strength in 3 *days* is easily obtained with Universal cement by applying the simple methods that have been demonstrated in thousands of laboratory tests made over a period of more than 5 years and used in actual construction practice. If quick-hardening concrete with high 1- and 3-day strengths is wanted, take advantage of *all* of the factors that contribute to making such concrete. These factors are:

"1. *Dry mixtures.* Use the *minimum* amount of mixing water.

"2. *Thorough mixing.* Never less than 1 min. and still better by additional time up to 5 min.

"3. *Rich mixtures*, which means an increased amount of cement. (Concrete A in the table on page 486 has 1.4 bbl., while the much stronger concrete F has 2.1 bbl. cement per cubic yard.)

"4. *Placing concrete* at a *temperature* of at least 70°F.

"5. *Maintaining* concrete at temperature of not less than 70° for the days indicated.

"6. *Curing.* Keeping concrete *damp* for the days indicated.

"7. Use of *calcium chloride* where tests show it gives an increased strength."

ELEMENTS AFFECTING CONCRETE CONSTRUCTION

Chart Presentation of Various Factors Given California Highways

BY C. S. POPE

Construction Engineer, California Highway Department

The following charts have been prepared for use of resident engineers and others with a view to predicting results which may be expected on different phases of concrete construction.

The data have been assembled from various sources which seem dependable but principal reliance has been placed on material available at this time from our own laboratory.

In common with all such generalizations, individual cases, due to the very numerous causes which may affect the results of tests of concrete materials may be expected from time to time to show results at considerable variance wit the findings of these charts.

Application of common sense and discrimination in their interpretation, however, will enable engineers to recognize the influence of these various factors and will make the charts of great value to those who have occasion to use them, and who properly interpret the data furnished by them.

Chart A.—This chart gives compressive strengths of sand and mortars of various proportions. The strengths are given in percentages of comprehensive strength of neat cement. A cement having a compressive strength, neat, of 12,000 lb. per square inch at 28 days is assumed as 100 per cent and all values given are prorated on the basis of the use of this cement in the varying proportions required.

The strength ratio shown assumes a sand which is structurally sound. A sand which is structurally weak will show higher strength ratios with a cement having a low neat strength than with a cement having a high neat

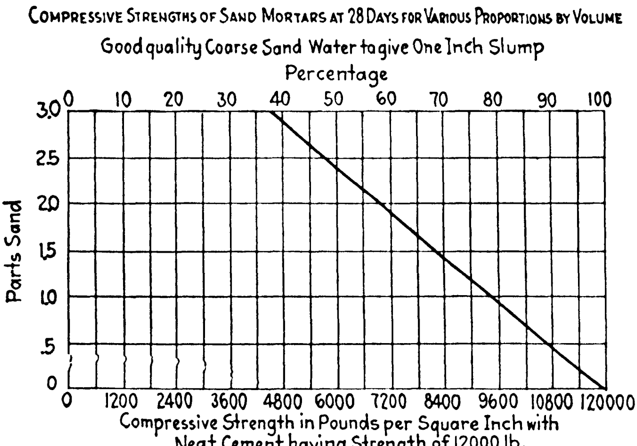


CHART *A*.

strength. The stronger cement brings out the weakness of the sand and the strength ratio for such a material would not be a straight line in the graph shown.

Sixty per cent is probably a conservative figure to use in estimating the probable strength of field concrete based on mortar strengths, the ratio of sand to cement in both concrete and mortar being the same.

Chart B.—This chart shows compressive strength of concrete with varying cement content. The compressive strength is shown in pounds per square inch based on first-class aggregate, and cement testing neat at 12,000 lb. per square inch at 28 days. The concrete is asssumed to have a ½-to-1-in. slump the water varying slightly as required by its aggregate.

Chart C.—This chart shows the effect of voids in coarse aggregate on compressive strength. The graph is particularly valuable and is shown both in percentages and in pounds per square inch. The effect of excess voids in the coarse aggregate is, of course, to require the addition of more sand to maintain workability with a corresponding decrease in compressive strength of the concrete.

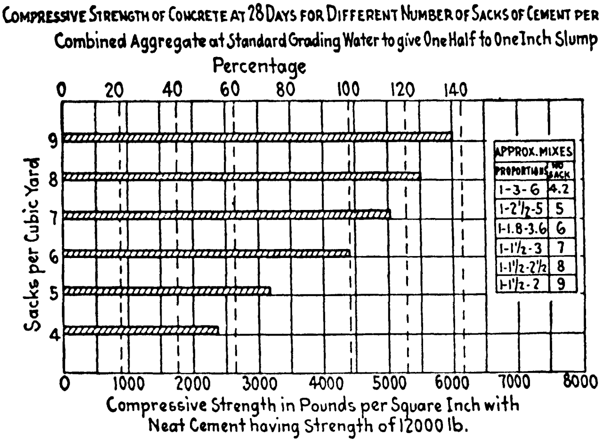


CHART *B*.

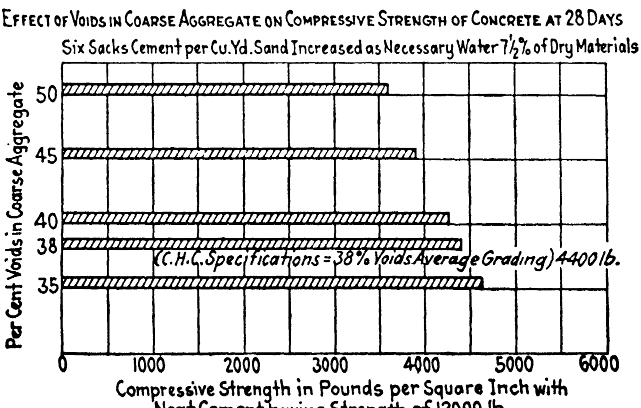


CHART *C*.

Water cement ratio is ratio of volume of water to volume of cement.

EFFECT OF WATER ON COMPRESSIVE STRENGTH OF SIX SACK CONCRETE
Aggregates at Standard Grading Twenty Eight Days

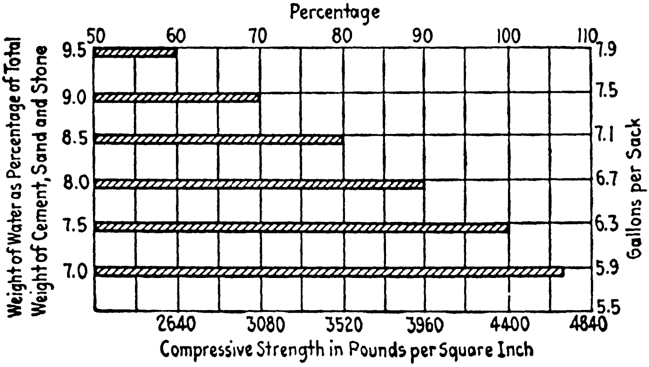


CHART *D*.

INCREASE IN COMPRESSIVE STRENGTH OF SIX SACK CONCRETE DUE TO TIME OF CURING
Combined Aggregate at Standard Grading Water to give One Inch Slump

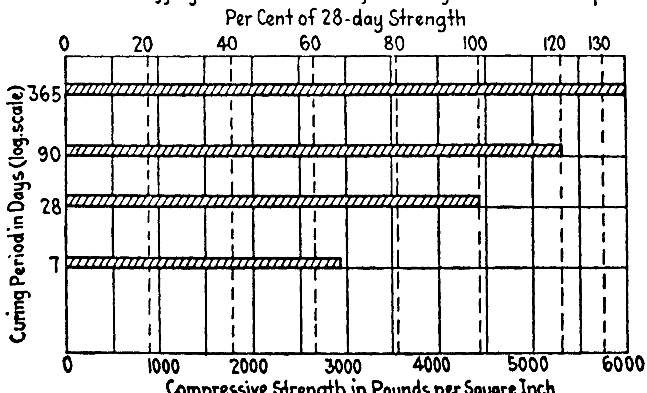


CHART *E*.

Chart D.—This table shows effect of water on compressive strength and is founded on tests in our own laboratory, on tests in Bulletin No. 58 of the Bureau of Standards, and on tests by Professor Abrams.

Chart E.—This chart shows increase in compressive strength with increased time of curing. The values shown will vary more with conditions than those given in other tables. In this instance the values are based on mean temperatures of 70 deg. or higher. Lower temperatures will retard curing time.

Bituminous-concrete Pavements

General Discussion.—Ordinary asphalt surfaces are probably familiar to most readers. An asphaltic-concrete surface is merely a well-graded mineral aggregate mixed and cemented with bitumen. This general type includes sheet asphalt, Topeka Mix, Warren Bros. Bitulithic, Amiesite, Bito slag, and various other trade-name products. These surfaces are laid on either macadam or cement-concrete bases. Specifications (p. 1459) describe construction operations in detail.

Asphaltic-concrete surfaces are satisfactory and economical for the heavier-traffic Class I rural roads having a traffic volume of over 3000 vehicles daily (10-hr. count) and for resurfacing of any road carrying over 1500 to 2000 vehicles daily. The essential advantage of this type lies in its pleasing appearance, ease of construction and repair, and resilient quality, which makes it particularly resistant to traffic damage of a large volume of mixed traffic.

The high cost of this surface eliminates its use on unimportant roads.

It has been extensively used for resident city streets and a considerable mileage has been constructed on primary state-system rural highways. In damp weather, it is often quite slippery for high-speed rural traffic—a drawback—but it is at present (1926) the most available type for resurfacing worn-out rigid pavements and for capping old solid macadam bases on roads of Class I and II.A, and it is certain that a considerable yardage of this type will be constructed on rural highways, particularly under reconstruction programs. Scientific grading of aggregates and care in construction do much to reduce the faults of waving and extreme slipperiness, although these fundamental faults are difficult to prevent entirely.

Asphaltic-concrete pavements, including concrete or macadam bases, cost for initial construction from $3 to $4 per square yard; the yearly maintenance has a wide range, but probably falls between 0.4 to 3.5 cts. per square yard for narrowrural high ways. The cost of surface renewal distributed over the life of the surface probably is about 8 to 15 cts. per square yard per year on narrow rural highway pavements (1926 cost conditions where the traffic volume is more than 3000 daily).[1]

The proper grading of aggregate is a complicated study and the reader is referred to books by Richardson, Hubbard, etc. for complete data. Specifications (p. 1459), show usual practice for rura

[1] Renewal costs based on maximum allowable Vialog coefficient of 250 without excessive maintenance (see p. 551).

highway pavements. The present tendency lies towards the use of increasing percentage of larger stone fragments under moderate-traffic volume to reduce slipperiness and the formation of waves. Under extremely heavy traffic, a fine aggregate is desirable to prevent pitting of the surface. A conservative estimate of surface life for this type on heavy-traffic rural roads is about 10 to 15 years, although there are many cases which do not fall within these limits (see Chap. VII).

Recommended Designs.—The recommended designs represent conservative practice. It is undesirable to reduce the thickness of base below the depths shown, as this type of pavement cannot be easily strengthened by future resurfacings. These designs are based on maximum gross vehicle load of 28,000 lb.

Suitability of Materials. *Macadam Bases.*—See Water-bound and Bituminous Macadam Pavements (pp. 429 to 449).

Concrete Bases. Cement.—Same as for concrete pavements (p. 456).

Coarse Aggregate. (Paving Base).—Any clean, hard, well-graded size of crushed stone, gravel, or 1800 lb. per cubic yard

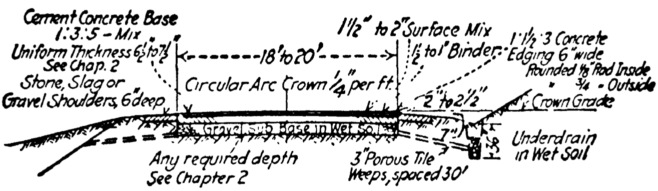


FIG. 165.—Recommended design Class I traffic.

crushed slag. Pit-run gravel or crusher-run stone should not be used; the sizing for coarse aggregate is generally specified as between ¾ to 3½", with not over 15% of material passing the 1¼" screen The high standards set for concrete pavement need not be attained, as the base is not subjected to traffic pounding. It must produce a strong concrete; it is desirable to use the best available source without raising the cost needlessly; in the main, a local supply should be preferred even if the coefficient of hardness drops as low as 5 for stone or this equivalent for gravel. Shale gravel shall not be used but a small percentage of shale up to 10% is allowable. The coarse aggregate must, however, be free from organic impurities and from a coating of clay or loam.

Sand (Paving Base).—Same general requirements as for concrete pavements except that an 8% maximum loam content can be used with safety (see Specifications, p. 1386).

Binder and Surface Mix—The following material from the "Asphalt Association Handbook" is as authoritative as any available data.

COARSE AGGREGATE—BROKEN-STONE SPECIFICATIONS

For Asphaltic Concrete, Coarse-graded Aggregate Type

Size	Per cent passing screen			
	1¼″	¾″	½″	¼″
Coarse aggregate........	95–100	25–75	......	0
Cover for seal coat......	......	.....	95–100	0–15

Per cent of wear, 5.

For Asphaltic Concrete, Fine-graded Aggregate Type

Coarse aggregate	Per cent passing screen		
	¾″	½″	¼″
Alternates			
I..........................	. .	95–100	0–75
II..........................	100	......	0–75

For Binder Course

	Per Cent
Passing 1¼″ screen..........................	95–100
Passing ¾″ screen..........................	25– 75
Passing ¼″ screen..........................	0– 10
Per cent of wear..........................	6

For Two-course Asphalt-macadam Base

	Per cent passing screen				
	3½″	2½″	1¼″	1″	¾″
No. 1 coarse...........	95–100	0–15			
No. 2 coarse...........	......	95–100	0–15		
Intermediate...........	......	......	95–100	25–75	0–15

Per cent of wear, 8.

For Asphaltic-concrete Base

Coarse aggregate, inches	Per cent passing screen				
	2½″	1½″	1¼″	¾″	¼″
Alternates:					
I..................	95–100	......	25–75	.....	0
II..................	......	95–100	.....	25–75	0

Per cent of wear, 5.

Broken-slag[1] *Specifications.*—Broken slag may be used in place of broken stone for the various types of asphalt pavements and bases, the requirements for size being the same as those given under Broken-stone Specifications. In addition, the following requirements are commonly made:

Per cent of wear....................	10 per cent −
Weight per cubic foot for each size specified........................	70 lb. +

Gravel[1] *Specifications.*—Gravel may be used in place of broken stone for coarse-graded-aggregate asphaltic concrete, binder coarse and asphaltic-concrete base with the same requirements for size as those given under Broken-stone Specifications. The gravel should be composed of sound, hard, durable pebbles free from clay or coatings of any character.

SAND OR FINE-AGGREGATE SPECIFICATIONS

COARSE-GRADED AGGREGATE ASPHALTIC-CONCRETE AND ASPHALTIC-CONCRETE BASE

Passing	Retained on	Per cent
¼" screen	....	100
¼" screen	10-mesh sieve	0-20
10-mesh sieve	40-mesh sieve	15-20
40-mesh sieve	80-mesh sieve	25-60
80-mesh sieve	200-mesh sieve	7-40
200-mesh sieve	....	0-6

FINE-GRADED AGGREGATE ASPHALTIC CONCRETE AND SHEET ASPHALT

Passing	Retained on	Per cent	
10-mesh sieve	....	98-100	
10-mesh sieve	20-mesh sieve	3-15	14-50
20-mesh sieve	30-mesh sieve	4-15	
30-mesh sieve	40-mesh sieve	5-25	
40-mesh sieve	50-mesh sieve	5-30	30-60
50-mesh sieve	80-mesh sieve	5-40	
80-mesh sieve	100-mesh sieve	10-20	16-40
100-mesh sieve	200-mesh sieve	6-20	
200-mesh sieve	....	0-5	

The sand shall consist of clean, hard, durable grains free from clay, loam, and other foreign matter.[2]

Mineral-filler Specifications.—Limestone dust, slate dust, or Portland cement.

	Per Cent or More
Passing 30-mesh sieve......................	100
Passing 200-mesh sieve....................	60

[1] Gravel or slag is not advised where a good grade of stone is available.

[2] Heat from drying drum generally eliminates all organic impurities except free matter like sticks or leaves.

Bitumen.—Detail specifications for suitable asphalt and tar binders are given on page 1399.

A considerable range in the value of penetration, depending on climatic and traffic conditions, is the only point necessary to indicate at this part of the discussion. The values recommended for asphalt are as follows:

TABLE 94.—PENETRATION VALUES FOR ASPHALTIC CONCRETE

Traffic	Temperature		
	Low	Moderate	High
Light	60–70	60–70	50–60
Moderate	60–70	60–70	50–60
Heavy	50–60	50–60	50–60

TABLE 95.—PENETRATION VALUES FOR SHEET ASPHALT

Traffic	Temperature		
Light	50–60	50–60	40–50
Moderate	50–60	50–60	40–50
Heavy	40–50	40–50	30–40

If a tar binder is used the usual grade corresponds to Tarvia X (Barrett Company Specifications).

Amounts of Material. *Macadam Bases.*—See Water-bound and Bituminous-macadam Pavements (pp. 441 to 449).

CEMENT-CONCRETE BASES PER CUBIC YARD

Mix	Coarse aggregate, cubic yards	Sand, cubic yards	Cement, barrels
1:3 :6	0.95	0.47	1.0
1:2½:5	0.92	0.46	1.2
1:2 :4	0.90	0.45	1.5
1:1½:3	0.84	0.42	1.9

Asphaltic Mixes.—The following data are taken from the "Asphalt Association Handbook" and applied to unpatented mixtures:

Coarse-graded Aggregate Asphaltic Concrete
(2″ thick, excluding seal coat)

Materials	Pounds per square yard	Tons per mile, 1′ wide
Coarse stone	147	43.1
Sand	58	17.0
Mineral filler	9	2.6
Seal-coat stone	25	7.3
Asphalt	18	5.3

Fine-graded Aggregate Asphaltic Concrete (Modified Topeka)
(2″ thick)

Materials	Pounds per square yard	Tons per mile 1′ wide
Stone chips	53	15.5
Sand	123	36.1
Mineral filler	18	5.3
Asphalt	18	5.3

Asphaltic-concrete Base
(3″ thick)

Materials	Pounds per square yard	Tons per mile 1′ wide
Coarse stone	216	63.4
Sand	100	29.3
Asphalt	18	5.3

"On western New York State Highways black base using a limestone aggregate and sand proportion approximately 75% stone and 25% sand, the weight per cubic yd. of consolidated material has ranged between 3500 and 3700 lb. exclusive of bitumen."

Amiesite.—This is a patented pavement made of crushed stone coated with asphaltic cement. It has been used on many miles of road with good results. It is shipped cold in a friable and granulated state, spread on either macadam or concrete base, and well rolled. Amiesite screenings are then spread and rolled, forming the surface. This construction costs about the same as asphaltic concrete per square yard. It resembles asphalt in appearance and has the advantages and disadvantages of all roads of this class. It is particularly adapted for small jobs where it would not pay to set up an asphalt plant or where suitable asphalt materials are not locally available.

TABLE 96.—DEPTHS AND WEIGHTS OF AMIESITE

(Weights are given per square yard)

	Amiesite loose, inches	Weight, pounds	Amiesite, square yards per ton	Filler loose, inches	Weight, pounds	Filler, square yards per ton	Total depth loose, inches	Total weight, pounds	Ultimate, compression, inches	Square yards per ton
Sandstone........	2¼	153	13.0	1	67¾	29.5	3¼	220¾	2	9.06
	2⅝	178½	11.3	1	67¾	29.5	3⅝	246¼	2¼	8.12
	3	204	9.8	1	67¾	29.5	4	271¾	2½	7.36
	3⅜	229½	8.7	1	67¾	29.5	4⅜	297¼	2¾	6.73
	3¾	255	7.8	1	67¾	29.5	4¾	322¾	3	6.20
Trap rock........	2¼	168¾	11.8	1	75	26.6	3¼	243¾	2	8.20
	2⅝	196⅞	10.2	1	75	26.6	3⅝	271⅞	2¼	7.36
	3	225	8.9	1	75	26.6	4	300	2½	6.66
	3⅜	253⅛	7.9	1	75	26.6	4⅜	328⅛	2¾	6.10
	3¾	281¼	7.1	1	75	26.6	4¾	356¼	3	5.60
Limestone........	2¼	164¼	12.2	1	73	27.4	3¼	237¼	2	8.43
	2⅝	191⅝	10.4	1	73	27.4	3⅝	264⅝	2¼	7.56
	3	219	9.1	1	73	27.4	4	292	2½	6.85
	3⅜	246⅜	8.1	1	73	27.4	4⅜	319⅜	2¾	6.26
	3¾	273¾	7.3	1	73	27.4	4¾	346¾	3	5.77

NOTES.—To find the amount of loose Amiesite necessary for any compressed thickness, subtract ½″ from compressed thickness, multiply by 1½, which equals loose thickness, to which add 1″ for filler.

To find what compressed thickness any given amount of loose Amiesite will give, subtract 1″ from loose thickness, multiply by ⅔, and add ½″ for filler.

Design.—Design includes decisions in regard to the selection of either macadam or cement concrete as the base, the depths of base required for the soil and traffic conditions prevailing, the width and crown of the pavement, and the mix and depth of the bituminous concrete surface.

Type of Base.—Macadam base is an economical design for moderate traffic where the macadam is constructed at least a year ahead of the asphalt surface. Macadam bases are rarely desirable where the asphaltic surface is laid on newly constructed macadam, as traffic pounding is necessary in order to compact macadam construction properly.

Cement-concrete base is the most feasible type for heavy traffic where it is desirable to get quick results on new grading, but should be used with caution on new fills of depths of over 2 to 3′ unless special care is taken in grading operations. Despite all care, it is not advisable to use concrete base on fills over 5′ deep or on side-hill sections part in cut and part in fill unless the grading is at least 1 year and preferably 2 years old.

Depth of Base.—If the macadam type of base is selected, it is merely a matter of macadam design. The required depths and the most economical use of local materials were discussed under

Macadam Design and the reader is referred to (pp. 366 to 392) and Macadam Pavements (pp. 429 to 449).

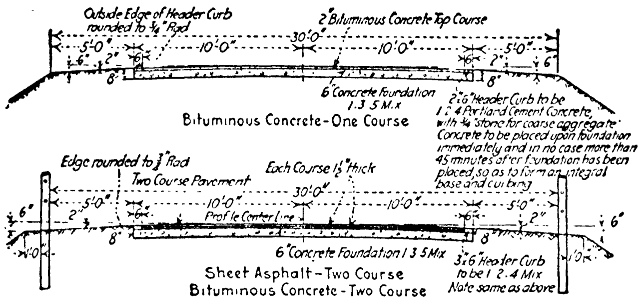


FIG. 166.—State of New Jersey (1922).

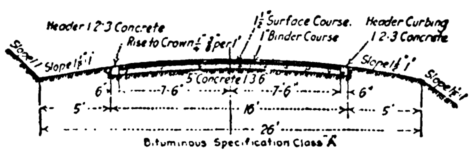


FIG. 166.—State of Pennsylvania (1922).

CONTRACTION AND EXPANSION OF MORTAR AND CONCRETE

Mortars		Concretes	
Mixture	Coefficient of expansion per degree Fahrenheit	Mixture	Coefficient of expansion per degree Fahrenheit
Neat	0.000,007,83	1:1½:3	0.000,006,77
1:1	0.000,007,43	1:2 :4	0.000,006,60
1:2	0.000,006,00	1:2½:5	0.000,005,58
1:3	0.000,006,05	1:3 :6	0.000,005,37
1:4	0.000,005,94		
1:5	0.000,005,77		

Coefficient of expansion. Steel reinforcement 0.000,006,5.

If the concrete type of base is selected, it is necessary to decide on the richness of the mix and the depth of base. Engineers differ as to the desirability of the different mixes, which range in common practice from 1:3:6 to 1:2:4. For the same strength, the leaner mixes require greater depth, and they are also a little more likely to be porous and to break down over trenches, but transverse contraction cracks seem less noticeable with the leaner mixtures, probably on account of lower coefficient of expansion and contraction. The tendency is towards the use of 1:2½:5 or 1:3:5 as a good practical mix. The depths of base recommended for different mixes under different traffic loading is given in Table 86 (p. 426) and ranges from 6 to 8″.

Surface Mix.—The surface mix is generally in two layers, binder and finer-aggregate mix. The depths range from 2″ where binder is not used to 3″ for the heavier-traffic streets (1 to 1½″ binder and 1½ to 2″ fine-aggregate top).

Figures 165 and 166 show the depths of binder and fine mix in common use (1926) on rural highways. Personally, the author prefers 1″ closed binder and 1½″ Modified Topeka Surface Mix for ordinary highway work. On extremely heavy-traffic road, a 1½″ binder with 1½″ of fine-graded mix is probably better for such conditions. There is an infinite variety of mixes, depending on materials available and local climatic and traffic conditions, and the best mix to adopt is largely a local problem which can be best solved by an expert asphalt man and gradually modified by observed action for any locality. Practically all authorities are agreed that the most common serious fault of surface mixes lies in insufficient fine sand passing 80 and being retained on 200 sieve, insufficient or poor-grade filler passing 200, and a poor relation between the proportions of fine sand, 80 to 200, to the amount of filler and bitumen. *The lack of durability of many pavements is directly chargeable to carelessness in grading.* The following typical mixes are recommended by the Asphalt Association and should serve satisfactorily in case local practice has not crystallized on a somewhat different mix. The reader is referred to books by Clifford Richardson, Provost Hubbard, Besson etc., for detail data on asphalt mixtures.[1]

TYPICAL MIXES RECOMMENDED BY ASPHALT ASSOCIATION

Asphaltic-concrete Surface Course

(Fine-grade aggregate type)

Thickness, 2″.
Crown, ¼″ to ⅜″ to the foot recommended.
Materials (see preceding discussion).
Mixture:

	Per Cent
Passing ½″, retained on 10-mesh screen	20.0–35.0
Passing 10-mesh, retained on 40-mesh sieve	7.0–25.0
Passing 40-mesh, retained on 80-mesh sieve	11.0–36.0
Passing 80-mesh, retained on 200-mesh sieve	10.0–25.0
Passing 200-mesh sieve	7.0–11.0
Bitumen (asphalt cement soluble in carbon disulphide)	7.5– 9.5

Sheet-asphalt Binder and Surface Courses

Thickness:

Binder course	1½″ recommended
Wearing course	1½″ recommended
Total	3″

Crown, ¼″ to the foot recommended.
Materials (see preceding discussion).
Mixtures:

[1] See Richardson, "Asphalt Construction," McGraw-Hill Book Co., Inc., p. 70. See Besson City Pavements McGraw Hill Book Co. Inc.

	Per Cent
Binder course:	
Coarse aggregate retained on 10-mesh sieve	60.0–80.0
Sand and material passing 10-mesh sieve	15.0–35.0
Bitumen (asphalt cement soluble in carbon disulphide)	4.0– 6.0
Surface course:	
Passing 10-mesh, retained on 40-mesh sieve	10.0–40.0
Passing 40-mesh, retained on 80-mesh sieve	22.0–45.0
Passing 80-mesh, retained on 200-mesh sieve	12.0–30.0
Passing 200-mesh sieve	10.0–20.0
Bitumen (asphalt cement soluble in carbon disulphide)	9.5–12.0

Probably the simplest way for the student to consider an asphalt mixture is in two steps: first, the asphaltic mortar composed of bitumen and all the mineral aggregate passing a No. 10 sieve, and, second, the coarse mineral aggregate retained on the No. 10 sieve. The mortar must conform to the requirements of a good sheet asphalt, all ingredients of which pass a No. 10 sieve, and the amount of coarse aggregate added to the mortar has no effect on proportioning the mineral aggregate of the mortar, although it does have some effect on the total bitumen in the final mix.

Good representative ideal mortar mixes used in northern states for different volumes of traffic are shown in the table on page 501.

It is important to have from 10 to 12% of bitumen to insure good life, waterproofing, etc., but in order to carry the proper amount of bitumen and still have a good workable and stable mix the proportion of bitumen, 200 and 80 factors, must be about as shown. A good grade of filler (200 sieve) is important. Limestone dust or Portland cement make the best and most dense product, but it is rarely advisable to eliminate the 200-mesh sand content entirely, although it is probably justifiable to restrict the 200-mesh sand to a maximum of 5% of mortar mix and make up the balance with the cement or stone-dust filler.

The second factor (coarse-stone aggregate 1⁄10 to 1⁄2″ in size) is added to increase stability of mix under traffic, to decrease slipperiness of the pavement, and to reduce the cost of the mix. For the ordinary so-called modified Topeka, which is a popular type, the amount of coarse stone ranges from 15 to 30 lb. per 100 lb. of total surface mixture. This coarse aggregate consists of the stone coated with bitumen. The amount of bitumen necessary to coat the stone of the size usually used is about 2% of the weight of the stone.

The final proportions of a mix consisting of both mortar and coarse aggregate can now be very easily computed. Suppose it is proposed to figure an ideal mix using 20 lb. of coarse aggregate and 80 lb. of mortar, making 100 lb. of completed surface mix. The amount of each factor in the mortar content of the final mix is obtained by multiplying the percentages given in the preceding table by 80 lb. The amount of bitumen figured for the mortar is then increased by 2% of the weight of the coarse aggregate (20 lb. $\times$ 0.02 = 0.4 lb.) and the net weight of coarse aggregate is added as a new factor. The table on page 501 of ideal mix for Topeka construction is computed in this manner:

ASPHALTIC-MORTAR MIX (PERCENTAGES BY WEIGHT)

Class of traffic	Bitumen, per cent	Mineral aggregate				Total all factors, per cent
		200 sieve	80	40	10	
Light	10.0	10.0	15	30	35	100
Medium	10.5	12.5	18	34	25	100
Heavy	11.0	15	21	34	19	100
Allowable variation (per cent each size or factor)	9.5–12	10–20	12–30	22–45	10–40	
	The relation of these columns must remain about constant, that is, they must all rise or fall together.			This column should not be much over twice the 80 column.	This column not so important.	

TABLE OF IDEAL PROPORTIONS, MODIFIED TOPEKA MIX
(Surface mix—percentages of complete mix by weight)

Class of traffic	Bitumen	Mineral aggregate						Total all factors
		200 sieve	80	40	10	¼"	½"	
Light	8.2	7.0	12	22	26	17	8	100
Medium	8.8	9.2	14	25	21	15	7	100
Heavy	9.5	11.5	17	28	16	13	5	100
Allowable variation (per cent for each size factor)	8–10	7–12	10–25	11–36	7–25	10–20	Less than 10	

Binder.—The binder course may be of either the so-called "open" or "closed" types. Open binder is merely asphalt- or tar-coated stone ranging in size from ⅝ to 1¼". This type of binder requires from 3.5 to 4.5% of bitumen. The closed binder is more nearly a true asphaltic concrete and requires careful grading of aggregate. By courtesy of Clifford Richardson the following is quoted from his book "Asphalt Construction."[1]

"**Open Binder.**—Open binder consists of stones largely of one size, the fragments being from ¾ to 1" in their largest diameter, although at times run of crusher has been used. Broken stone of the latter description requires a larger amount of bituminous cementing material, and in consequence the binder is more cohesive. The following data will show the average composition of open binders which have been used at different times in the construction of sheet-asphalt pavements.

"It appears from these figures that the percentage of bitumen which a binder requires depends largely upon the amount of fine material which it contains. The first mentioned in the table contains 26.8 % of fine material

[1] "Asphalt Construction," McGraw-Hill Book Company, Inc.

AVERAGE PERCENTAGE COMPOSITION OF OPEN BINDERS USED IN SHEET-ASPHALT PAVEMENT

	Test No.									
	69978		70804		70854		71102		74893	
Bitumen	5.4		4.4		3.8		3.6		3.5	
Filler	5.8	26.8	4.1	16.6	2.2	9.7	2.4	5.4	1.5	4.5
Sand	21.0		12.5		7.5		3.0		3.0	
Stone:										
Passing 1/4" sieve	5.8		8.7		18.0		13.5		49.5	
Passing 1/2" sieve	13.6	67.8	46.8	79.0	52.0	86.5	51.5	91.0	10.0	92.0
Passing 1" sieve	41.4		23.5		16.5		26.0		32.5	
Retained 1" sieve	7.0		0.0		0.0		0.0		0.0	
	100.0		100.0		100.0		100.0		100.0	

and requires 5.4 % of bitumen, while those made from cleaner stone where the finer particles do not exceed 5%, contain less than 4% of bitumen. In preparing, sending to the street, and placing a course of open binder, care is demanded in certain directions. The stone should be hard, sufficiently so as not to crush under the roller. It should be free from clay and dirt, although, as has been said, fine particles of the stone itself are an advantage rather than otherwise. In heating the stone and mixing it with the asphalt cement, great care should be used so that it is not overheated, since if this is the case a proper coating of the bonding material will not adhere to the stone, owing to the excessive heat, or much of it may run off and be lost during the haul to the street. On the other hand, the binder should be hot enough to permit of properly coating it and its ready compression on the foundation. An open binder should be bright and glossy, and not dead in appearance, as it is dumped from the truck. It should be hot enough to spread readily and uniformly. It should contain no excess of bitumen at any one spot, and, should this be the case, such spots should be removed and replaced. Binder, after it has been placed on the street or road, should be covered, within the shortest space of time possible, with the surface, as when it has been wet, tracked with dirt, or become covered with horse droppings or dead leaves, the adhesion of the surface mixture to it will not be complete.

"In actual practice a 9-cu. ft. box of broken stone for binder will weigh about 900 lb., will require about 40 lb. of Trinidad and about 36 lb. of Bermudez asphalt cement, but the cement must be regulated by observing the appearance of the material in the truck and on the street.

"In preparing a binder at the plant the mixer in use should have teeth with a sufficient clearance between them and the lining of the mixer so that the largest particles of stone cannot become wedged between them, and thus rapidly wear out the lining of the mixer. It is not good practice to attempt to mix binder in the same mixer that is employed for preparing the surface mixture. A separate one should be employed, although in some of the smaller plants of a portable description it is not always possible to do so, in which case provision must be made for making a change of teeth from the longer to a shorter form, in changing from surface to binder mixing."

Close Binder.[1]—Close binder is, or should be, a true asphaltic concrete. It consists of the same broken stone of which the open binder is made, but the voids in it are filled with smaller stone and with an asphaltic mortar corresponding to the ordinary asphalt surface mixture. For the construction of a close binder or asphaltic concrete of the highest type, the grading of the mineral aggregate should be carefully regulated. This can be done by determining the voids in the coarser stone, calculating the amount of fine stone necessary to fill these, and again the amount of sand to fill those in the mixture of coarse and fine stone. This can be readily done by constructing a box of sheet iron or wood of exactly 1-cu. ft. capacity. It is filled with the hot, coarse stone, which is compacted by shaking. The surface is then struck off and the box weighed. Allowing for tare, the weight of 1 cu. ft. of the coarse stone is obtained. Knowing the density of this stone, 2.65 for limestone and 2.9 to 3.0 for trap rock, the weight of a solid cubic foot of the stone can be calculated. By dividing the weight of a cubic foot of the broken stone by the weight of a solid cubic foot of the material, the voids can be determined. In the same way the weight of a cubic foot of the finer stone and of sand can be arrived at. The amount of each which it is necessary to use to fill the voids in the coarse stone, and again in the mixture of the coarse and fine stone, can be readily calculated. The proportions for actual use can be readily determined and the amount of bitumen required from the percentages of broken stone and sand that are present.

As an example of the proportions of coarse stone, fine stone, sand, and asphalt cement in actual use in preparing a close binder, the following figures will serve:

	New York		Boston
	Plant 1	Plant 2	
Coarse stone..	480 lb. = 54.6%	480 lb. = 53.3%	
Fine stone....	200 lb. = 22.7%	202 lb. = 22.4%	885 lb. = 73.4%
Sand...	150 lb. = 17.0%	150 lb. = 16.7%	250 lb. = 20.8%
Trinidad asphalt cement	....	68 lb. = 7.6%	70 lb. = 5.8%
Bermudez asphalt cement	50 lb. = 5.7%		
	880 lb. = 100.0%	900 lb. = 100.0%	1205 lb. = 100.0%

ANALYSES, PER CENT

Bitumen soluble in CS_2.....	5.6	....	5.2	....	4.8	
Passing 200-mesh screen...	4.4	....	6.4	....	4.2	
Passing 10-mesh screen.....	29.2	33.6	29.0	35.4	26.4	30.6
Passing 8-mesh screen.....	1.8	....	2.0	....	0.8	
Passing ¼″ screen..........	10.0	....	7.0	....	4.4	
Passing ½″ screen..........	23.2	....	25.8	....	24.4	
Passing ¾″ screen..........	13.6	....	17.8	....	32.2	
Passing 1″ screen..........	7.4	....	6.8	....	2.8	
Retained 1″ screen..........	4.8	....	0.0	....	0.0	
	100.0	....	100.0	....	100.0	

"At some plants it is impossible to separate the stone into coarse and fine particles, in which case the heated stone of both sizes is collected in one bin, but segregation generally occurs under such circumstances. In arranging the grading of the mineral aggregate, care should be taken to see that the

[1] Quoted from RICHARDSON, CLIFFORD, "Asphalt Construction," McGraw-Hill Book Company, Inc.

amount of fine material, while sufficient to fill the voids in the stone, is not present in excess, since in this case the surface of the binder when compacted on the street will be too smooth to bond properly with the wearing surface, and resulting displacement under travel will be possible. An excess of bitumen must also be avoided for the same reason, and if spots in the binder as laid show anything of this description, the material should be removed.

"An entirely satisfactory close binder can be constructed for all but the most trying conditions by the use of selected old surface material, which has been disintegrated and softened by steam, for filling the voids in the ordinary open binder. It is, of course, a matter of great economy to turn out a close binder in this manner. The old surface material, having been crushed and softened by dry steam, is added in the mixer to the hot binder stone and, in this manner, further disintegrated and distributed among the stone. The proper amount of additional asphalt cement is added to coat the stone and enrich the old surface mixture. It does not possess the complete stability of an asphaltic concrete, but it is infinitely superior to the ordinary open binder for the construction of an intermediate course.

"The asphalt cement in use in binder may well be considerably softer, 10 to 15 points, when it is possible to make it so, than that in use in the sheet-asphalt surface.

"In the construction of asphalt pavements with an asphaltic concrete binder course, the surface should be applied to it before it has become entirely cold. It is well to run out binder to the street for not more than half of a working day and to cover it with surface during the same day. This is much more important with a close than with an open binder It is not possible to do satisfactory work where a large area of close binder is laid on one day and covered with surface on the next."

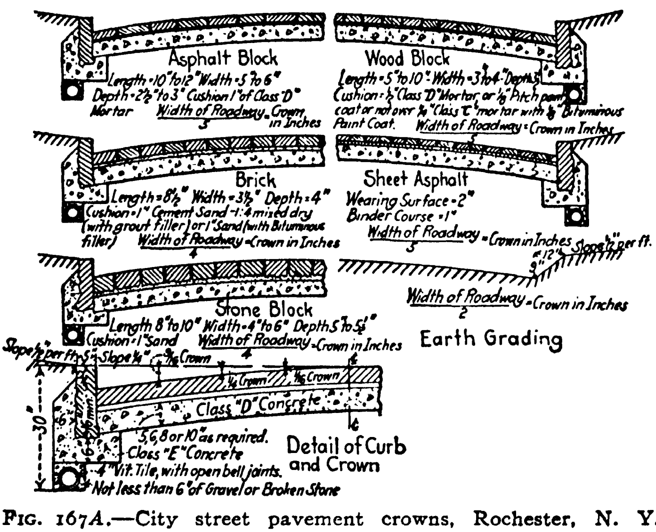


FIG. 167A.—City street pavement crowns, Rochester, N. Y.

Width and Crown.—Normal widths for heavy-traffic roads on which asphaltic concretes can be economically used range from 18 to 20′ on straight alignment. On sharp curves these widths are increased (see Tables 36 and 37, p. 132).

Either the parabolic or circular-arc crown is suitable; as discussed under Concrete Pavements, the circular arc is to be preferred. A crown slope of ¼ to ⅜" per foot is usual practice. For rural highways, ¼" is probably the better rate. For village streets of considerable width, Fig. 167 page 505 shows current practice for different types of pavement. As the grade increases the crown is reduced. On sharp curves, a one-way banked crown is used (see p. 127 for ordinary practice).

Steep Grades.—See pages 425 and 101 for limitations of use imposed by steep grades.

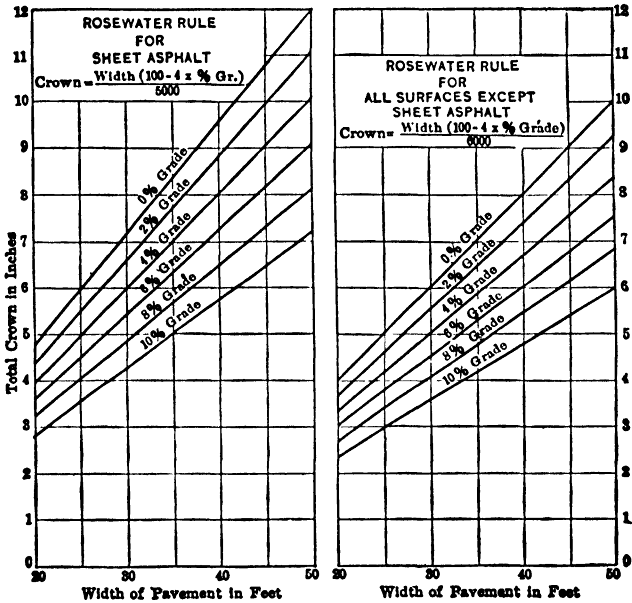


FIG. 167*B*.—City street crowns, "Rosewater Rule."

Specifications.—See Part III (pp. 1459 to 1470).

Construction Equipment.—See Chap. XV (pp. 1264 to 1265).

Inspection Details.—See Chap. XVI (pp. 1313 to 1330).

General Maintenance Methods.—Usual methods of shoulder, ditch, and guard-rail work prevail. The pavement is repaired by cutting out the depressions or waves which develop and replacing the surface mix; this can be easily and quickly done with very little inconvenience to traffic (see Chap. VII for costs, etc.). **Temperature cracks are sealed with liquid bitumen.**

Brick Pavements

General Discussion.—All readers are familiar in a general way with the character of brick pavements. They are merely specially hard, well-manufactured brick set on either a cement-sand, bituminous-sand, or sand cushion and the joints between the brick filled with cement grout, bituminous filler, or, in unusual conditions, plain sand. The foundation for the brick surfaces is generally cement concrete ranging in depth from 5 to 8″. In some cases firm macadam foundations can be utilized to advantage in place of the cement-concrete base. Specifications (p. 1476) describe construction operation in detail.

Brick pavements are satisfactory for the heavier Class I traffic roads. They have been used extensively for city pavements and for a considerable mileage of primary state highways. The economy of their use depends largely on the location of the road in relation to the brick-manufacturing plant, as the item of freight has of late years been the cause of a serious curtailment of the use of this type of road. There have been cases where brick roads close to brick plants cost about as much as at greater distances, but this is an artificial condition due to lack of competition and to the manipulation of sale price, situations which are likely to develop with any material. Well-manufactured paving brick is an excellent surfacing material, as the pavement is pleasing in appearance, having a warm, neutral tone; it is a safe surface for high-speed travel; it absorbs very little moisture and has no tendency to push into humps or waves. For rural highway conditions, however, the initial cost is usually so much higher than other acceptable types that it is rarely selected on the score of economy under moderate traffic, and where it is used (largely for portions of rural roads passing through village business streets) the selection is based on the factors of extremely heavy traffic, appearance, and local preference for the type. It seems a pity that such an excellent material has been so badly handicapped by freights and by an adherence to a form of brick which may perhaps be modified to advantage for rural highway conditions. The size of the proposed highway program demands the use of all available materials, and clay products are so widely distributed that it seems ridiculous not to make more use of this excellent material. Some attempts have been made to modify brick-pavement practice to make it more applicable for the ordinary rural road. These attempts have not been entirely successful but show sufficient promise to warrant more serious consideration of modifications in practice. The cube method is described on page 516. Reduction in thickness of paving brick is receiving attention and there is considerable evidence of successful progress along this line of modifying old style practice particularly for moderate traffic conditions.

Standard 4″ brick pavements, including base, cost, for initial construction, from \$3.50 to \$5.00 per square yard, 1926 cost conditions; the yearly maintenance has a wide range due to all sorts of conditions, but probably falls between 0.3 to 4 cts. (1.5 average) per square yard. The cost of surface renewal distributed over the life of the pavement is probably about 10 to 16 cts. per square yard per year for rural highway conditions on primary roads carrying over 4000 vehicles daily.[1] A reasonable surface life for heavy-traffic roads constructed of this type is probably about 12 to 18 years, although there are many cases which do not fall within these limits (see Chap. VII).

There are three general types of brick pavement:

1. Monolithic base and top with cement-grout joint filler.
2. Semimonolithic type with cement-concrete base, cement-sand, or bituminous-sand cushion and either cement-grout or bituminous joint filler.
3. "Rolled base" with some type of macadam or gravel base surfaced with brick having either bituminous or cement-grout joint filler.

The monolithic type is constructed by laying the brick on the green concrete base having a skim coat of cement sand before the base has set; in this way a fairly good bond between base and top course of brick is obtained. The purpose of this construction was to produce a monolithic slab, and by this means reduce the necessary depth of concrete base. It has not proved as effective as its originators hoped. The semimonolithic is the best-known construction; it depends largely for its strength on the concrete base, which is constructed and allowed to set before the surface is constructed. On this base an evener cushion of sand or a mixture of sand and cement or bitumen and sand is spread, and the brick surface laid and filled with either cement grout or bitumen. The "rolled-base" type is essentially the same as the semimonolithic, except that macadam or gravel is substituted for the cement-concrete base.

The monolithic type should be used with caution, in northern climates at least, as it does not seem to meet conditions as well as the second class. The rolled-base type should be used with caution under extremely heavy traffic, and even under moderate traffic it has some disadvantages, due largely to the shape of the standard block. The best type of brick pavement appears to be the second class noted above.

If the second class (semimonolithic) is used, there is considerable disagreement between engineers in regard to the use of cement-sand cushion as against bituminous-sand cushion and in regard to the use of cement-grout filler as against bituminous-mastic filler. Assuming that the base is strong enough to prevent failure due to weakness of the pavement and that the brick are uniformly well manufactured and stand the traffic pounding, the surface defects which develop are due either to temperature cracking or blow-ups or spalling at joints. Cracking and blow-ups are reduced by the use of bituminous filler in place of grout filler; joint spalling is reduced by the use of cement-grout filler. Excellent pavements can be constructed by either method. In villages where appearance is a large factor, and where it may be necessary to take up areas of pavement, the author personally favors the cement-sand cushion with bituminous joint filler. On

[1] These renewal costs are based on a maximum permissible Vialog rating of 250″ per mile without excessive ordinary maintenance charges.

rural highways outside of villages where appearance is not such a large factor, the author prefers the cement-sand cushion with cement-grout filler, as, while a small percentage of cracks will occur, the pavement is not seriously damaged its riding quality for high-speed traffic is bettered and joint wear reduced. Recent tests on the Bates Road (1922) seem favorable to the bituminous sand cushion. Recent tests (1926) U. S. Office of Public Roads are summarized as follows.

Conclusions Indicated by the Investigation.—The several parts of the investigation have now been completed and the analysis of the data obtained seem to warrant certain conclusions, among which the more important are:

1. That 2½-in. brick of the quality used in the Arlington traffic tests, when properly supported, will prove satisfactory for pavements carrying the heavier types of traffic.
2. That brick of 2-in. thickness, when properly supported, and of the quality used in the tests, will be adequate for pavements on streets carrying the lighter types of traffic.
3. That a bedding course of plain sand is more effective in reducing breakage of brick than a cement-sand bedding course, the breakage being much less on the former than the latter. The depth of the sand bedding course should not greatly exceed ¾ in. Increasing the depth tends to produce roughness in the pavement.
4. That cobbling of the brick is greatly increased as the spacing between bricks is increased.
5. That the use of excessive quantities of asphalt filler is a common and serious fault in construction, unnecessarily increasing the cost and resulting in a condition which impairs both the appearance and the serviceability of the pavement.
6. That base construction of other than the rigid type may in many cases prove entirely satisfactory. Macadam bases and those constructed of certain types of natural earth appear to be suitable when the local conditions are such that these types of construction maintain their stability throughout the year.
7. That no difference in the base construction is necessary for the different thicknesses of brick.

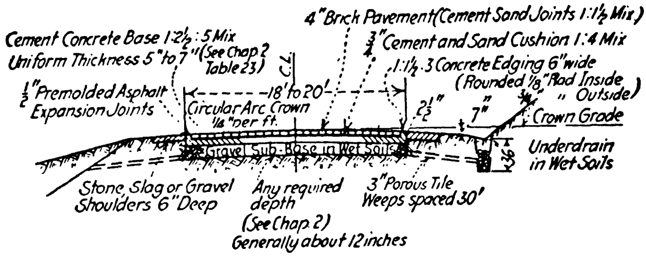
Considering the fact that brick surfaces are an expensive type which can only be economically justified under heavy traffic, it is most certainly desirable to use only the best of materials in the construction. The depth of the base should be amply strong when first constructed, as it is poor policy to take a chance on depth failure for this type. Materials, details of design, and inspection are outlined on page 508.

Brick-paving practice is going through a transition stage in the same way as concrete-paving practice, due to the radical difference in service demanded by modern motor traffic on rural highways as contrasted with the old horse-drawn vehicles, and any statement of recommended practice is tentative at best.

Recommended Designs.—The following designs can be used with an assurance of reasonable success. They represent conservative practice for localities having a gross vehicle-load limitation of 28,000 lb.

Suitable Materials.—Suitable materials for macadam and cement bases are the same as discussed under Macadam Pavements (pp. 429 to 449) and under Asphaltic-concrete Pavements (p. 492).

Brick (Size).—The size of standard paving block is usually specific as between 3¼″ wide, 3¾″ deep by 8½″ long and 3½″ wide by 4″ deep by 9″ long. For any particular size used on single jobs, the maximum difference in depth should not exceed ⅛″, in width ⅛″, and in length ½″. Kiln marks over 3/64″ in height cause the rejection of individual brick. If the brick have rounded edges they should not exceed 3/16″ radius (square brick are preferable where a cement-grout filler is specified). Wire-cut or pressed lugs on one side of the brick should produce a separation of the block of at least ⅛″ and not over ¼″. This standard size produces a very effective and handsome pavement on village or city streets and on Class I traffic rural highways. There is, however, a growing need for a smaller unit for resurfacing worn-out



168*A*.—Recommended design (cement grout joints.)

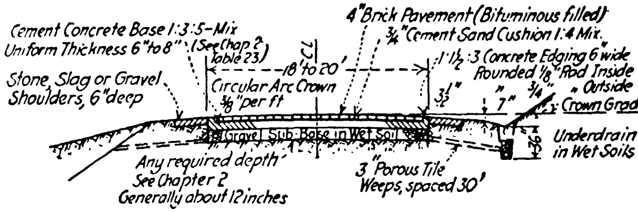


FIG. 168*B*.—Recommended design (bituminous joints).

macadam and concrete pavements, particularly on grades of more than 4% where asphaltic-concrete pavements are not entirely satisfactory because of the tendency of motors to skid in damp weather. Whether this need will be met by small stone blocks or by thin brick or brick cubes is still in doubt and will depend very largely on the general attitude of the stone-block and brick-paving manufacturers in producing an acceptable grade of cube at a reasonable cost. The standard 4″ block is not only very clostly for such conditions but also introduces too much change in surface elevation, particularly where a contracted section in cut with gutter is used.

A considerable yardage of pavement constructed of brick of standard surface dimensions but with a 3″ depth has been laid in an effort to reduce freight costs on the materials. There is considerable doubt as to the advisability of reducing depth dimensions where the width and length remain the same, as it unbalances the resistance of the block, particularly over a macadam base, and for heavy-traffic roads this expedient may well be used with caution. Special-shaped, grooved blocks, known as hillside brick, are used on steep grades.

Quality.—Paving block are manufactured of special mixtures of clay or shale. Most pavers are a shale product, although very good brick can be manufactured by a well-controlled clay mixture. The essential qualities of the finished product are that the brick shall be well annealed, tough and evenly burned, and free from laminations, voids, and cracks. When broken, they must show a uniform, stone-like fracture. There are two general types, the wire-cut brick and the repressed brick. The consensus of opinion among engineers seems to favor the wire-cut method, as the brick is, perhaps, less likely to show laminations. The wire-cut block also has the square edge, which is desirable where cement grout is used as the joint filler, as there is less liability for the grout to chip out.

Where bituminous joint filler is used, a well-manufactured wire cut brick without lugs is generally specified.

All paving materials are subject to tests for toughness and absorption, and in some cases to the rupture test for cross-breaking, although the last test is hardly necessary for the standard-size block. Toughness is determined by the rattler test described on page 715. The maximum loss by abrasion in this test is usually set at 24% for heavy-traffic roads and the maximum absorption at 3½%. Modulus of rupture is to be not less than 2000 lb. per square inch computed by the formula

$$R = 3\frac{WL}{2bd^2},$$

in which R = modulus of rupture.
W = load in pounds which produces rupture.
L = length between supports (6″).
b = width of brick, in inches.
d = depth of brick, in inches.

Cushion Sand.—The plain sand cushion is going out of use in favor of either the cement-sand or bituminous-sand cushion. Where a sand cushion is used, it must be free from pebbles and fairly clean. A common specification calls for 100% passing a 6 sieve, 90% passing a 20 sieve, and not over 10% of loam or silt. Unsatisfactory portions of many brick pavements have been directly traceable to poor cushion and to careless manipulation, which resulted in the sand working up between the brick and preventing proper grout penetration.

Where the cement-sand cushion is used, a first-class concrete sand containing not over 8% of loam is required; the usual mix for this type of cushion is 1 part cement to 4 of sand, which gives the necessary stiffness to prevent flowing of the cushion.

Where bituminous-sand cushion is used, the sand should be a clean, coarse concrete sand mixed with about 4 to 6% of bitumen of the proper grade (see page 1399).

Grout Sand.—Must be a clean, sharp concrete sand containing not over 5% of loam or silt. A common specification requires 100% passing a 12 sieve and not over 40% passing a 50 sieve. A mixture of 1 part cement to 1 part grout is often specified. The National Brick Manufacturers recommend 1 part cement to 1½ parts sand.

Bituminous Filler.—Either asphalt or tar-pitch filler can be successfully used for Specifications (see page 1401).

Amounts of Materials.—The amounts of material for macadam bases are given under Macadam (p. 441), and for concrete bases on page 495.

Brick is usually purchased by the square yard finished pavement on account of culling, etc., so that the number of brick per square yard is of little value. In case brick is purchased by the 1000, the following table from Blanchard will be of use.[1] Ordinary standard paving block 3½ by 4 by 9″ lay about 40 to the square yard.

Size of brick	Number of brick per square yard ⅛″ joint
2¼ by 8 by 4 laid flat...........	67
2¼ by 8¼ by 4 laid edgewise.......	65
3¼ by 8½ by 4 laid edgewise............ ...	45
2½ by 8½ by 4 laid edgewise..........	57
3 by 9 by 4 laid edgewise........	46

Grout.—One barrel of cement will grout approximately 36 sq. yd. of standard 4″ depth block with grout mixed 1:1.

Expansion Joints.—Premolded joints purchased by the linear foot are usually used. In case the joints are poured, 1 bbl. of paving pitch will fill approximately 130 lin. ft. of joint 1″ wide and 4″ deep.

Design.—Design covers selection of type of pavement (monolithic, semimonolithic, or rolled base), depth of base, type of brick surface (grouted or bituminous filled), type of cushion (cement sand, bituminous sand or plain sand), width, and crown of pavement.

Type.—As previously discussed, the semimonolithic on cement-concrete base is the type probably entitled to most consideration for heavy-traffic rural highways.

Depth of Base.—If a macadam base is used, the design of depth is quite definite, as discussed on page 391. If the cement-concrete type of base is used, any existing formula is speculative to say the least, and experience is the best guide. Experience can, however, be expressed in the form of tentative formulas, as developed on pages 421 to 427.

[1] BLANCHARD, "Highway Engineering."

Adequate depths of macadam base under heavy traffic depend on the foundation soil and range from 8 to 24″, using Formula (2), (p. 384), with a wheel load of 14,000 lb. (28,000-lb. gross static vehicle load) and assuming that a bituminous-filled brick top is equivalent to 2″ of macadam.

Adequate depths of cement-concrete bases depend on the richness of the mix of the concrete base, on whether the pavement is of the monolithic or semimonolithic type, and, if of the semimonolithic type, whether the surface is grout filled or has a bituminous filler.

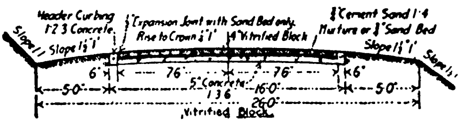


FIG. 169*A*.—Standard brick pavement. State of Pennsylvania (1922).

Current practice generally uses a 4″ depth of cement concrete with 4″ brick for the monolithic type, and for the semimonolithic type, with concrete ranging in mix from 1:3:6 to 1:2:4, the depth of base ranges from 5 to 8″ with slightly greater depth for bituminous-filled brick. Tentative formulas developed on page 421 indicate that for rural highway conditions (28,000-lb. gross vehicle load), using the recommended semimonolithic type with a base mix of 1:2½:5, a depth of 6 to 7″ for grout-filled surface and 7 to 8″ for bituminous-filled surface will be satisfactory.

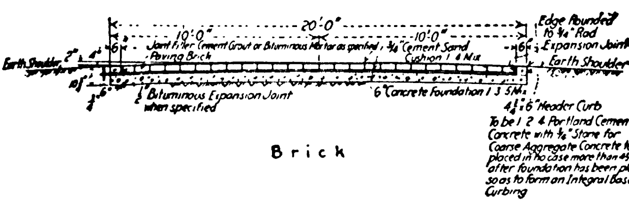


FIG. 169*B*.—Standard brick pavement. State of New Jersey (1922).

Expansion and Contraction.—Bituminous-filled brick surfaces do not require any special provision for temperature stresses.

Cement-grout-filled brick require longitudinal joints along curb or edging. Premolded bituminous strips are the best type of joint. A joint width on each side of ½″ is sufficient for rural roads up to 20′; for wider streets, 20 to 30′, ¾″ width; and for greater widths, 1″.

Transverse joints are rarely used and appear to be detrimental; infrequent blow-ups may better be permitted. Where these joints are used they are generally spaced at 50′ intervals and have a ½″ thickness.

Width and Crown.—Widths are the same as for any rigid rural pavement (pp. 130 and 6). Widening on curves is desirable. Where a state highway with a large volume of traffic passes through villages, 30′ is satisfactory for resident streets and 40 to 60′ for business streets. The crown usually used for 18 to 20′ rural roads is a circular arc, ¼″ per foot for grouted brick and ⅜″ per foot for bituminous filled; banking on curves is given (p. 127). Village-street crowns are indicated (p. 505). The straight-line crown is to be avoided, as discussed on page 474 (Concrete Pavements).

Steep Grades.—Hillside brick should be used on grades over 5% for grouted surfaces and on grades over 8% for bituminous-filled brick where horse traffic must be considered.

Specifications.—See Part III (p. 1476).

Equipment.—See Chap. XV.

Inspection Details.—See Chap. XVI (p. 1330).

General Maintenance Methods.—See Chap. VII (p. 569).

Asphalt-block Pavement

Asphalt-block pavements are an excellent type for extremely heavy Class I traffic, particularly where roads of this class pass through villages and the grades are not steeper than about 4%. The pavement shows a smooth, uniform surface, dustless, and practically noiseless. Its life probably lies between 15 and 20 years, depending on the quality of block. Failures that occur in shorter time are generally due to inferior block. The quality of block is important. A mist or light rain makes the pavement very slippery on grades of over 4%. Within a reasonable freight radius of the point of manufacture this pavement costs about the same as brick for original construction. It is used as a surfacing on both concrete and macadam bases. This type of pavement should be used with caution on macadam bases, as it is an expensive surface and its form is not so applicable to use on top of macadam as the sheet-asphalt type.

The asphalt-block pavement laid on New York State rural highways has been very satisfactory. The proportion of ingredients is about 70% crushed rock, usually trap, which has passed a ¼″ ring, about 20% limestone dust to act as filler, and approximately 10% of asphaltic cement, molded under a pressure of 2 tons per square inch of block having a 2″ depth This produces a dense asphalt much superior to the ordinary sheet. The asphalt used is Trinidad. This is refined and fluxed so that the resulting asphaltic cement may be varied as to adhesiveness, penetration, etc., to meet varying conditions peculiar to different localities. The penetration is made high enough to give a certain amount of pliancy to the block, to avoid crumbling at the edges, and to make the joints self-healing.

The use of blocks containing steel anchors, laid across the road approximately 15′ apart, has eliminated any movement of the block under traffic. These blocks are placed at more frequent intervals on curves. Block pavements have been laid, using a longitudinal row of these anchor blocks in place of edging. The results appear satisfactory, although under these conditions it is probably desirable to thicken the base at the edge.

After the base is prepared a mixture of 1:4 Portland cement mortar is spread ½″ thick. This mortar bed is carefully screeded and the block laid thereon, joints being broken at least 4″.

An interesting comparison with brick occurs in the "pinning in" curbs. Instead of bats being broken by hand a large mechanical shear is used. Each fractional block is measured and cut to fit exactly.

After being laid, the pavement is given a light coat of sharp sand, which is broomed into the joints. Traffic is permitted in 4 or 5 days.

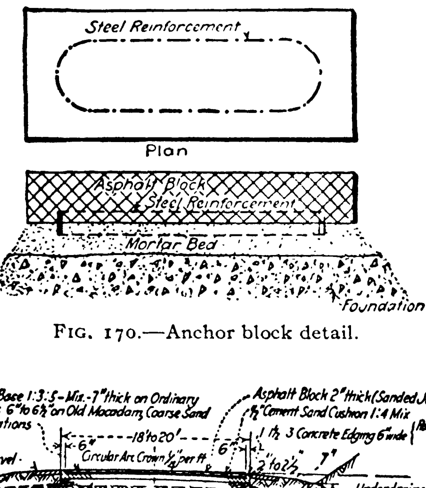


FIG. 170.—Anchor block detail.

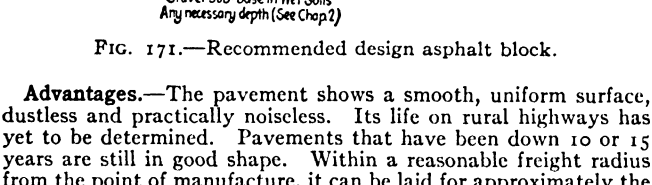


FIG. 171.—Recommended design asphalt block.

Advantages.—The pavement shows a smooth, uniform surface, dustless and practically noiseless. Its life on rural highways has yet to be determined. Pavements that have been down 10 or 15 years are still in good shape. Within a reasonable freight radius from the point of manufacture, it can be laid for approximately the cost of brick.

Disadvantages.—A mist or light rain makes the pavement very slippery. It should not be used on grades of over 4%.

Recommended Design.—The following recommended design is for conservative practice in localities having a gross vehicle-load limitation of 28,000 lb.

Suitable Materials.—Materials for concrete base are the same as discussed under Asphaltic Concrete (p. 492).

The block itself, being a manufactured product generally laid under a guarantee, cannot be too definitely described in a general specification. It is, perhaps, just as well to specify trap-rock stone aggregate where this is available, as such blocks apparently wear considerably better than the softer limestone blocks. Well-manufactured blocks using stone aggregate with a per cent of wear of 5 or less will probably be satisfactory; the consistency of the bituminous binder had better be left to the manufacturer so long as he understands the traffic and climatic conditions to be met. An example of the ordinary specifications is given page 1474.

Amounts of Materials.—Amounts for bases are the same as given under Asphaltic Concrete Pavements.

Design.—The design of base thickness, width, and crown is the same as for asphaltic concretes (p. 497).

Equipment and General Methods.—Brick-paving equipment and methods apply.

Inspection Details.—Inspection is essentially the same as for brick-pavement work.

Specifications.—See pages 1474 to 1476.

LESS-USED PAVEMENTS

The text on the following pavements is condensed, as they are at present used for comparatively small yardage under special conditions. The specifications give essentials (see page 1479).

Wood-block Pavements

Wood block can be rarely used to advantage on rural highways. In exceptional cases they can be considered. Specifications are given on page 1479.

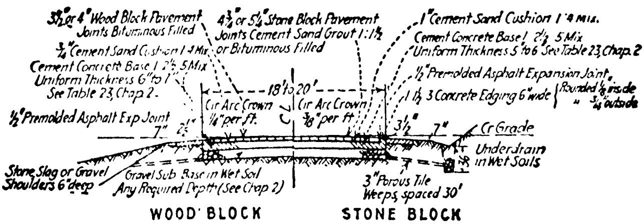


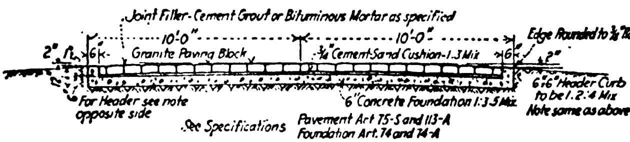
FIG. 172.—Recommended design. Stone and wood block.

Stone-block Pavements

Stone-block pavements, on account of their high first cost and somewhat irregular surface, are uncomfortable for high-speed traffic. They are generally specified on rural highways only on steep-grade Class I traffic roads where the normal pavement adopted is too slippery for safety. For these conditions these pavements offer the best possible and most economic solution, considering service and final cost. Specifications are given on page 1481. The following recommended design is conservative practice for Class I roads.

Small Stone-block Surfacing.—In Germany, Hungary, Austria, and England a surfacing made of granite blocks, ranging in size from 2½″ to 4″, has been used successfully. This pavement is known as "Kleinpflaster" in Germany and as "Durax" armoring in England. The stone cubes must be cut with considerable accuracy in order to give a smooth and durable surface.

The blocks are laid on a thin sand cushion of about ⅜″ depth, on either a macadam or concrete foundation; they are thoroughly rammed to give a firm bearing and the joints filled with either clean



STONE BLOCK

FIG. 173.—Standard section, State of New Jersey (1922).

sand flushed in or a bituminous filler. The joints do not exceed ¼″ in width. The courses of cubes are laid either diagonally to the direction of the traffic or in concentric rings.

Where the stone is broken by hand the cost is high, and it would be impossible to consider its use for rural roads in this country. A machine[1] has, however, been developed in Europe for breaking these cubes which is claimed to produce a satisfactory product at a reasonable rate. It is a belt-driven friction drop hammer having a stone chisel mounted on the anvil; the hammer head is shaped like a stonecutter's sledge. About 1½ hp. is needed for each machine.

About 400 of these machines are in operation, and a plant in Sweden is turning out 700,000 sq. yd. of pavement per year with 62 machines.

Provided the stone-cube surface, exclusive of base, can be laid for $3 to $4 per square yard, it seems a type that must be seriously considered on steep-grade Class I traffic, particularly for resurfacing old pavements under reconstruction programs. A price as low as this, however, would probably necessitate the use of convict labor in the manufacture of the cubes.

McClintock Cube Pavements

This is a patented pavement devised by J. Y. McClintock, County Engineer of Monroe County, New York. The patent has run out

[1] A detailed description of this machine is given in *Eng. News*, Mar. 27, 1912.

and it is available for use without the drawbacks of patent limitations. It is very similar to "Kleinpflaster," except that under his patent artificial cubes as well as stone cubes are proposed. It appears to be a very promising type.

The construction is essentially as shown in Fig. 174 and consists of a top course of 2¼" cubes placed on a thin sand, cement-sand, or bituminous cushion supported by either a macadam or a concrete base. The cubes have been made of concrete, vitrified-paving-brick material, and stone as in Continental practice.

They are loaded, hauled, and dumped like broken stone; laid in close contact by means of a pallet and rake, 128 at a time on a sand cushion ¼ to ½" thick, no care being taken to break joints. They are then rolled to bring to an even and firm bearing; the joints are filled with a sandy loam and the surface treated with a light coat of light road oil or cold tar if the foundation is macadam. The joints are grouted if the foundation is concrete. Temporary shoulders of 2" plank are put down during the laying of the cubes, after which they are removed and replaced with broken stone or gravel as shown in Fig. 174.

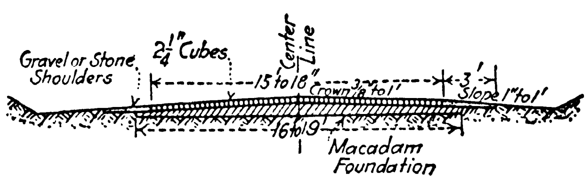


FIG. 174.—Typical section cube pavement on macadam base.

The experience of the past 13 years has shown that this form of construction using a sand-tarred joint is flexible under frost action, which makes it suitable as a surfacing on a macadam base. It keeps its shape under traffic and shows no tendency to ravel or break down at the edges, and can be successfully held with a macadam or gravel shoulder without the formation of a rut along the edge, which is a difficulty always encountered where a rigid edging is designed. It gives a satisfactory surface in both wet and dry weather and can be laid late in the season. The cubes require comparatively little inspection and can be successfully used as patches in maintenance with simple manipulations. They reduce the tonnage and freight cost where imported materials are required. The cubical form simplifies laying and makes a well-balanced block to resist rupture. This type is essentially the macadam type, with a perfect-shaped macadam unit, the cube. Concrete cubes have not served satisfactorily, failing in spots, but this is to be expected, as it is not a reliable material for a road surfacing of this nature (that is, for such small units). Vitrified-shale cubes with wide sand joints laid on a macadam base have shown ability to stand medium traffic. Vitrified-shale cubes with close tarred joints laid on a thick macadam base serve very satisfactorily under moderately heavy traffic, and these cubes laid on a concrete foundation and grouted meet all but the heaviest traffic satisfactorily.

Consider briefly the present tendencies in highway construction. There are two distinct types: the flexible form, represented by the macadams, and the rigid types, such as brick, asphalt, stone block, etc., having concrete foundations. Each has a distinct field and their relative economy depends largely on the traffic.

It is sufficient for this discussion to note that macadams are suitable for light and medium traffic (Classes II and III), that they are able to withstand climatic changes better than the rigid pavements, and that with a moderate yearly expenditure they can be kept in good condition when used under the volume of traffic stipulated.

They fail either under high-velocity traffic or heavy hauling, the first being a surface failure and the second a foundation failure for most of the roads, but a surface failure for some which have a thick, well-consolidated base. That is, if some better flexible surface can be used on a first-class macadam foundation, this type of road will be able to handle a heavier volume of traffic than at present with a moderate maintenance charge. The indications are that the brick cubes with sand-oiled joints will serve this purpose.

The rigid roads develop defects due to temperature changes, frost heave, and the settlement of fills. Subsequent movement is localized along these lines, and eventually expensive repair and reconstruction is necessary. Under heavy traffic, however, the cost is less than for the macadam type and the inconvenience of continual repairs is avoided.

The first cost of standard rigid pavements is so high that designers often hesitate to use them where they are actually needed. If it were possible to reduce the cost and yet obtain practically the same class of improvement a larger mileage could be used to advantage.

The indications are that the brick cubes on a concrete foundation will serve this purpose at a cost considerably less than the present paving brick.

Highway designers do not hesitate to use macadam for the light traffic roads or expensive rigid constructions for the extremely heavy traffic; the great mileage that lies on the verge of either form of construction offers the real difficulty. It is for this class of road that the cubes are particularly adapted by reducing the cost of brick and increasing the efficiency of macadam. This applies also to the resurfacing of concrete and macadam roads which on steep grades is an annoying problem to the highway engineer.

It is believed that, provided this type fulfills its present indications, it will meet a recognized need in highway construction, particularly in reconstructive programs. For this reason more space than perhaps is justified has been given to a method which has not been tested out by a large-mileage construction.

This type of surface compares favorably in initial cost with asphaltic concretes. It is adaptable to manufacture by convict labor.

CHAPTER VII

MAINTENANCE

Maintenance is more a matter of business organization than of technical engineering knowledge. Reconstruction is primarily an engineering proposition (see Chap. VIII). Effective maintenance methods and the reasonable uses of different materials for repair have been quite well determined by practice. Existing flaws in maintenance practice are due largely to insufficient funds and political interference with the personnel of the maintenance organization. Of these two difficulties, lack of funds is the most serious. Funds are a legislative tax problem and a proper solution always lags behind the necessities of the situation. For years, engineers have warned legislative bodies that more adequate provision must be made for repair, but active steps to provide such funds have been slow in developing. This is natural, as it is difficult for a non-technical man to appreciate the necessity before he has some physical evidence to support the statement, and such evidence is only furnished by roads which have gone to pieces. Modern conveniences cost money, and no expedient has yet been devised to avoid "paying the piper." At the present time, the necessity for more adequate sums is receiving popular attention and support in states having improved systems of an average age of 10 years or more, and there is every indication that remedial action will be taken. Improved highways have demonstrated their worth so thoroughly that there is no danger of any community permitting them to continue permanently in poor condition. Reasonable legislative action depends on engineering estimates of maintenance and renewal costs.

MAINTENANCE AND RENEWAL COSTS

There are two general classes of estimate:

1. For drafting a general legislative tax program.
2. Yearly budget estimates.

The first type of estimate requires consideration of future conditions.

The second class states definitely the immediate requirements of the situation. The first class of estimate is subject to considerable uncertainty on account of fluctuating labor and material costs, change in traffic volume, spasmodic maintenance, control of truck loading and speed, etc., but a reasonably close approximation can be derived by an experienced maintenance man. The yearly budget estimates are an easy, definite problem, as they are based on actual physical evidence.

TABLE 97.—PROBABLE AVERAGE MAINTENANCE AND RENEWAL COSTS FOR DISTRICTS SIMILAR TO WESTERN NEW YORK

(COMPILED IN 1922)

(Computed for normal conditions. Exceptionally favorable conditions in regard to maintenance appropriations and personnel of the force might easily reduce maintenance and renewal costs 20 to 30%—see Table 105)

Type of pavement	Width, feet	Average depth, inches	Construction costs: Contract cost per square yard pavement only	Construction costs: 5% interest on cost	Yearly maintenance: Pavement only per square yard	Yearly maintenance: Shoulders, ditches, guard rail, etc.	Renewal[a]: Assumed normal life	Renewal[a]: Yearly cost for renewal	Total maintenance and renewal: Per square yard	Total maintenance and renewal: Per mile	Total interest on construction, maintenance, and renewal: Per square yard	Total interest on construction, maintenance, and renewal: Per mile
Column No. 1	2	3	4	5	6	7	8	9	10	11	12	13
Class I traffic (2000 or more vehicles daily 10-hr. count in summer)												
Stone block on concrete base	18–20	12	$5.50	$0.275	$0.005	$0.020	30–35	$0.100	$0.125	$1,400	$0.400	$4,400
Brick on concrete base	18–20	11	4.20	0.210	0.015	0.020	15–20	0.160	0.195	2,150	0.405	4,450
Asphaltic concrete on cement base	18–20	10	3.50	0.175	0.020	0.020	10–15	0.170	0.210	2,300	0.390	4,300
Reinforced cement concrete[a]	18–20	8	3.20	0.160	0.015	0.020	12–15	0.160	0.195	2,150	0.360	4,000
Penetration bituminous macadam[a]	18–20	13	2.50	0.125	0.070	0.020	5–9	0.200	0.290	3,200	0.420	4,600

Class II traffic (800–2000 vehicles daily 10-hr. count in summer)												
Reinforced cement concrete[a]	16–18	8	$3.20	$0.160	$0.010	$0.015	15	$0.145	$0.170	$1,700	$0.330	$3,300
Asphaltic concrete on macadam base	16–18	12	3.00	0.150	0.020	0.015	12	0.150	0.185	1,850	0.335	3,350
Brick cubes on macadam base	16–18	12	3.00	0.155	0.020	0.015	12	0.160	0.195	1,950	0.345	3,450
Penetration bituminous macadam	16–18	12	2.20	0.110	0.040	0.015	10	0.120	0.175	1,750	0.285	2,850
Water-bound macadam (oiled)	16–18	12	1.90	0.095	0.080	0.015	8	0.110	0.205	2,050	0.300	3,000
Class III traffic (300 to 800 vehicles daily 10-hr. count in summer)												
Penetration bituminous macadam	12–16	10	$1.90	$0.095	$0.030	$0.012	12	$0.100	$0.140	$1,100	$0.240	$1,900
Water-bound macadam	12–16	10	1.60	0.080	0.060	0.012	10	0.070	0.140	1,100	0.220	1,800
Class IV traffic (less than 300 vehicles daily)												
Water-bound macadam	8–12	9	$1.50	$0.075	$0.040	$0.012	12	$0.060	$0.110	$ 650	$0.180	$1,100
Gravel[b]	8–12	10	1.00	0.050	0.040	0.012	8	0.030	0.080	500	0.130	800

[a] These types eventually resurfaced with asphaltic concrete or some form of standard block or cube surface.

[b] This type can be temporarily used under heavy traffic, but for a volume of over 400 to 500 daily it generally develops disagreeable waves known as rhythmic corrugations.

[c] Based on maximum allowable roughness of 250″ per mile Vialog measure without exceeding normal maintenance expenditures.

Legislative Tax Estimates.—Reliable estimates of the future cost of maintenance and renewal depend more on personal knowledge and personal records of an experienced engineer than they do on the exact application of official published reports of maintenance costs. Students may well bear in mind that the usual official report on maintenance expenditure is a broad average, subject to misinterpretation due to inaccuracies of bookkeeping, inadequate strength of some of the pavements, spasmodic appropriations for maintenance, difference in age of pavements, difference in volume of traffic, etc. If due allowance is made for these factors, the values finally adopted often depart considerably from published figures.

Table 97 (p. 520) gives a basis for future cost estimates which can be used with assurance of reasonable accuracy for conditions similar to those in New York State. This is a conservative estimate, assuming average conditions of efficiency. If the roads are designed of adequate strength and are maintained under an unusually effective system of appropriation and personnel, the maintenance and renewal costs could probably be reduced about 20 to 30% (see Table 105, p. 536). The data were derived as follows:

In order to avoid needless inaccuracies as much as possible, this discussion is based on careful analysis of the records of Division 4, New York State Department of Highways, of which the author has personal knowledge, supplemented by general State Reports of New York and Massachusetts. The data are analyzed in two ways: (1) Reported average costs are modified for average conditions of age, strength, volume of traffic, and effectiveness of maintenance. This analysis gives a good basis for future estimates under prevailing conditions. (2) Typical individual roads are selected which are known to be adequate in strength and have received moderately good attention; the costs for these special roads are tabulated. This method indicates what can be done under good supervision with adequate funds. Division 4 is selected for analysis as it has been used throughout this book to illustrate the practical application of theories of finance, type selection, traffic volume, etc.

Yearly Costs per Vehicle Mile.—The Highway costs given in Table 97 expressed in terms of vehicle miles may be roughly approximated as follows:

TABLE 97*A*

Class of traffic	Assumed average daily volume	Average cost of maintenance and renewal per vehicle mile, cents	Average cost of interest maintenance and renewal per vehicle mile, cents
Class I	4000	0.2	0.3
Class II	1500	0.3	0.6
Class III	600	0.5	0.8
Class IV	200	0.7	1.4

The saving in traffic operation cost due to well-maintained, hard-surfaced roads is probably about 1 to 1.5 cts. per ton mile, which indicates that modern improved roads can probably be justified by economic travel return for volumes of traffic of over 300 daily, but that for traffic of less than this amount other more indirect benefits must be considered to justify hard-surfaced, year-round pavements (see Fig. 3, p. 16, Chap. I).

Basic Conditions, Division 4, New York State Department of Highways.—This division includes six counties in western New York having an area of 3600 sq. miles, an assessed valuation, 1920, of approximately $523,000,000, a total motor vehicle registration, 1920, of 58,000, and a total road mileage of 6700 miles. At present there are 870 miles of state-improved roads under maintenance and the following analysis is based on official cost reports for this mileage, modified by personal knowledge of conditions. The executive personnel of this division has been excellent; it is as free from objectionable political interference as any locality is likely to be. There is no great frequency of overloaded truck traffic. The main difficulty has been lack of adequate funds. Table 98 shows at a glance that, due to lack of funds and average age, the maintenance costs as reported must be modified if used as a basis for long-time legislative programs. Table 97 shows the necessary modified values.

Independent Check on Essential Correctness of Table 97 for Legislative Estimate Purposes.—Table 97 was built up from a detailed study of yearly maintenance figures and records of pavement durability. In order to check these results, the following test was made approaching the problem from an entirely different angle based on independent data. The essential agreement of the two methods indicates that the statistics given in Table 97 provide a safe basis for territory similar to western New York. The following tabulation and footnote are self-explanatory.

Tabular Comparison of Reasonable Estimated Requirements for Maintenance and Renewal Based on Table 97 (Class II Traffic) as Compared with Actual Appropriation Expenditures

(Division 4, New York state highways, average traffic conditions Class II)

Year	Miles of road under maintenance	Table 97 normal requirements per mile per year	Estimated normal yearly requirements, Table 97	Actual appropriations for maintenance and renewal
1914[1]	510	$1,100	$ 600,000	$ 360,000
1915[1]	590	1,200	700,000	376,000
1916[1]	660	1,300	900,000	330,000
1917	690	1,600	1,100,000	606,000
1918[2]	720	1,700	1,200,000	1,071,000
1919	750	1,800	1,400,000	674,000
1920	780	1,900	1,500,000	874,000
1921	840	1,800	1,500,000	1,080,000
1922	860	1,800	1,600,000	1,200,000
Totals	...	....	$10,500,000	$6,571,000

[1] Table 97 modified for prewar price scale.
[2] Emergency war appropriation (army-truck damage).
This table indicates that in 8 years actual appropriations have lagged behind reasonable upkeep requirements $4,000,000.

Table 98 (p. 526) shows that in January, 1923, the reconstruction program for worn-out surface is 7 years behind normal renewal for water-bound and 4 years behind normal renewal for bituminous macadams, and that to bring the system up to reasonable standard 300 miles should be immediately resurfaced. Such work will cost approximately $14,000 to $22,000 per mile, or a total of approximately $5,000,000, which checks the deficit in appropriations obtained by the application of Table 97 to this district.

The data are analyzed (pp. 523 to 534) for effect of strength, age, and traffic volume. The use of Table 97 in connection with legislative tax estimate (Division 4) is illustrated for the benefit of the student on page 20, Chap. I.

Analysis of Maintenance Costs. *Traffic Classification* (Table 99).—The basic factors of maintenance costs are volume and character of traffic. Table 99 classifies the roads by a study of the short-time traffic counts modified by common sense for future traffic growth (see p. 528).

Class I traffic—2000 or more vehicles daily (10-hr. count in summer).

Class II*A* traffic—1500–2000 (10-hr. count in summer).

Class II traffic—800–1500 (10-hr. count in summer).

Class III traffic—300–800 (10-hr. count in summer).

The outstanding conclusions from Table 99 are as follows:

1. The maintenance data on the macadam types for this district should be fairly conclusive, as there is enough mileage at a great enough age to mean something.

2. The maintenance cost for the rigid types is not yet at all conclusive and is probably far under normal, considering age and traffic volume.

3. Water-bound macadams are in the main, serving slightly more traffic than is recommended for this type.

4. Bituminous macadam is apparently serving about the proper volume for this type.

5. Cement concrete is serving considerably less volume than is recommended for this type.

6. Brick is serving somewhat less volume than is desirable for this type.

7. Stone block has been selected for footing on steep hills. Volume of traffic has no bearing.

8. Asphaltic concretes are, in the main, serving somewhat less than a reasonable volume of traffic for this type.

Strength of Existing Pavements, Division 4.—According to the best judgment of the engineering force in the division, based on observation of the resistance of the existing pavements to traffic action, the strength of the different types may be classed as follows:

Water-bound macadam, noticeably weak due to a large mileage of old, thin roads.

Bituminous macadam, moderately good due to a larger per cent of more recent design.

Asphaltic concretes on macadam, moderately good.

Asphaltic concretes on concrete, adequate with a few exceptions.

Cement concrete (1:2:4 mix or better), slightly low.

Brick on concrete, adequate.

Stone block on concrete, adequate.

These results appear in column 3, Table 98 (p. 526).

Recent designs which approximate the recommended depths developed in Chap. VI seem adequate for traffic.

Effect of Traffic Volume.—Records in this regard are meager. Table 100 (New York State) shows a start along these lines, but the figures are not very convincing, as records of soil conditions, adequate original strength, and relative age of pavements are not considered and the finance bookkeeping methods were not devised with the idea of enough subdivisions to give extremely accurate records for a tabulation of this kind. It is therefore necessary to use considerable judgment in assigning rational values to this factor of cost. According to the best judgment, a conservative set of yearly maintenance costs under different volumes of traffic for the most used types is given in Tables 97 and 98. One of the outstanding features of Table 100 is the high cost of shoulder maintenance on Class I traffic, indicating that 16′ width of pavement is poor economy under such traffic.

TABLE 98.—BASIC CONDITIONS DIVISION 4, JANUARY, 1923—MAINTENANCE COSTS (RENEWAL COST NOT CONSIDERED) (*Compiled by W. G. Harger for illustrative purposes*)

Type of pavement	Total mileage (1)	Physical modifying conditions					
		Average traffic conditions (2)	Actual pavement strength (3)	Actual average age, years (4)	Estimated reasonable average age for completed system provided renewals were properly made (5)	Per cent mileage needing resurfacing at present (6)	Normal yearly per cent resurfacing, long-term basis (7)
Gravel	5	Class III −	Adequate	6.1	5 years Class III (minus)	44	10
Water-bound macadam[a]	370	Class II −	Low	8.3	5 years Class II (minus)	70	10
Penetration bituminous macadam	244	Class II	Moderate	8.0	6 years Class II	35	8
Asphaltic concretes (macadam base)	25	Class II +	Moderate	5.0	6 years Class II	5	8
Asphaltic concretes (concrete base)[b]	36	Class I −	Adequate	6.6	7 years Class I	20	7
Cement concrete (1:2:4 mix or better)	124	Class II +	Moderate	2.4	7 years Class II (plus)	1	7
Cement concrete (1:2½:5 mix)	20	Class II	Moderate	5.0	3 years Class II	100	16
Brick (concrete base)	25	Class I	Adequate	7.5	9 years Class I	0	6
Stone block (concrete base)	2	Class II	Adequate	6.5	15 years Class II	0	3
Brick cubes (macadam base)	10.3	Class II	Low	13.0	7 years Class II	100	7

Column 5 is one-half of normal life which expresses normal conditions for average age figures.

$$\text{Column 7} = \frac{100}{\text{Normal life}}.$$

NOTE.—The outstanding feature of this tabulation is the large percentage needing resurfacing, which shows that the funds for renewals have been consistently inadequate and that ground is being lost each year, as regards the general condition of the system.

[a] Columns 3 to 7 would indicate that the reported maintenance cost should be high for water-bound macadam. As a matter of fact actual expenditures are low, as the maintenance budget estimates are consistently reduced below the amount required, as the funds are short. The main roads are given preference and the water-bound type (largely on secondary roads) have to get along with less expenditure than they are entitled to.

[b] The large per cent requiring renewal at a short age is due to poor original construction (overburned mix).

TABLE 98—*Continued*

Type of pavement	Effect physical conditions shown in columns 2 to 7 should have on necessary costs (8)	Approximate average cost maintenance pavement per year (5-year average) (9)		Remarks	
		Per mile	Per square yard	Adequacy of actual expenditures (10)	Finally adopted actual relation of reported maintenance costs to normal long-time condition, considering existing appropriation limitations as well as physical conditions prevailing
Gravel	High	$500	$0.065	Low	Normal
Water-bound macadam[a]	High	580	0.065	Low	Normal (minus)
Penetration bituminous macadam	High	400	0.043	Slightly low	Normal (plus)
Asphaltic concretes (macadam base)	Normal (minus)	120	0.012	Adequate	Normal (minus)
Asphaltic concretes (concrete base)[b]	Normal (plus)	160	0.017	Adequate	Normal
Cement concrete (1:2:4 mix or better)	Very low	40	0.004	Adequate	Very low
Cement concrete (1:2½:5 mix)	High	450	0.048	Low	Normal
Brick (concrete base)	Normal (minus)	80	0.009	Adequate	Normal (minus)
Stone block (concrete base)	Very low	75	0.008	Adequate	High
Brick cubes (macadam base)	High	300	0.032	Low	High

TABLE 99.—APPROXIMATE MILEAGE AND AGE UNDER DIFFERENT TRAFFIC CLASSIFICATIONS, DIVISION 7, JANUARY, 1923

Traffic classification	Total miles	Average age	Mileage at different ages, years (January, 1923)																				
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	20	21
Gravel pavement (bituminous seal coat)																							
Class I																							
Class IIA																							
Class II																							
Class III	5.3	6.1	...	...	...	...	...	2.4	...	2.9													
Water-bound macadam (bituminous seal coat)																							
Class I	4.5	12.7	...	...	...	...	1.0	...	...	...	...	...	0.2	...	2.0	...	...	...	...	...	...	...	1.3
Class IIA	51.5	8.5	...	...	...	...	5.5	5.3	9.3	10.0	...	...	1.0	...	15.1	3.5	0.3	1.5	...	...	...	...	...
Class II	177.3	8.4	...	...	8.9	0.3	17.3	12.0	21.9	21.5	17.9	...	14.4	10.3	5.2	11.4	16.9	13.5	4.5	...	...	1.3	
Class III	158.3	7.1	23.3	3.3	...	11.6	3.4	10.4	24.3	20.3	15.5	...	0.5	2.7	10.5	18.2	7.9	2.7	3.7				
Penetration bituminous macadam																							
Class I	33.9	7.9	.	.	...	0.2	8.9	3.6	1.9	...	3.2	...	8.8	...	7.3								
Class IIA	45.8	8.0	...	...	0.4	2.6	10.6	...	1.0	1.4	0.8	1.4	12.1	10.5	5.0								
Class II	105.6	7.7	12.6	3.0	2.7	5.4	1.2	3.4	3.6	8.2	11.5	8.2	1.2	28.8	14.1	1.7							
Class III	52.9	9.0	...	0.3	...	0.5	2.7	1.2	3.4	1.4	...	19.2	11.6	8.7	2.2	1.7							
Asphaltic concrete, macadam base (total mileage not entirely covered in this table)																							
Class I	4.1	7.5	...	0.2	...	...	0.5	0.8	...	1.5	...	...	...	...	1.1								
Class IIA	9.6	5.0	...	0.3	0.7	2.4	...	4.6	...	0.6	...	0.1	...	...	0.9								
Class II	2.8	6.8	...	0.9	...	...	...	0.8	...	...	...	...	...	...	1.1								
Class III	0.6	3.0	...	0.4	...	...	...	...	0.2														
Asphaltic concrete, concrete base																							
Class I	8.5	6.4	...	...	2.3	1.1	...	...	...	0.7	1.4	...	3.0										
Class IIA	12.9	5.0	...	3.6	...	...	...	4.1	1.4	...	...	3.8											
Class II	5.9	8.0	...	0.3	...	...	...	...	1.0	...	...	4.6											
Class III	10.9	9.0	...	...	...	...	...	...	...	...	...	10.9											

Cement concrete, 1: 2: 4 mix or better																							
Class I........	9.5	3.0	...	1.6	2.6	....	3.1	2.1															
Class IIA.....	38.5	4.0	1.1	15.6	....	0.3	....	9.0	....	....	12.5												
Class II.......	31.8	1.0	2.1	20.7	6.3	2.7																	
Class III......	35.0	1.4	3.4	22.5	1.5	4.2	3.4																
Brick on concrete base																							
Class I........	8.7	8.0	...	...	....	...	...	1.0	0.1	1.7	3.1	2.4	....	....	0.4								
Class IIA.....	6.1	7.5	...	....	..	....	....	....	1.1	2.0	2.6	0.2	0.2										
Class II.......	6.1	8.0	....	....	....	....	....	....	0.6	....	4.6	....	....	....	....	....	0.9						
Class III......	1.8	8.0	....	....	....	....	....	....	0.6	....	0.6	0.6											

TABLE 100.—TABLES SHOWING AVERAGE COST OF MAINTENANCE OF VARIOUS TYPES OF PAVEMENT CLASSIFIED ACCORDING TO THE AMOUNT OF TRAFFIC

(1918 to 1922 inclusive New York State)

Amount of traffic per 12-hr. day, summer months	Miles	Cost of maintenance per mile per year	
		Pavement only	Total maintenance
Gravel pavement			
Less than 500	110.76	$584	$737
500–1000	31.49	721	924
1000–2000	6.62	675	872
Over 2000	0.60	824	983
Total	149 47 Average	$622	$785
Water-bound macadam			
Less than 500	1111.72	$551	$658
500–1000	763.26	652	843
1000–2000	360 12	602	897
Over 2000	38 51	881	1110
Total	2273 61 Average	$615	$766
Bituminous macadam, penetration method			
Less than 500	947.41	$303	$429
500–1000	1050 61	355	499
1000–2000	798 03	409	612
Over 2000	317 09	646	889
Total	3113.14 Average	$382	$547
Mixed bituminous macadam on macadam base			
Less than 500	12.87	$375	$473
500–1000	5.90	513	612
1000–2000	7.48	302	484
Over 2000	17.89	544	913
Total	44.14 Average	$449	$673
Mixed bituminous macadam on concrete base			
Less than 500	7.66	$ 99	$300
500–1000	22.65	146	231
1000–2000	30.39	146	208
Over 2000	29.02	229	336
Total	89.72 Average	$169	$293
Hassam pavement and second-class concrete			
Less than 500	47.79	$532	$759
500–1000	84.71	495	675
1000–2000	57.97	545	768
Over 2000	21.88	474	728
Total	212.35 Average	$517	$724

TABLE 100—*Continued*

Amount of traffic per 12 hr. day, summer months	Miles	Cost of maintenance per mile per year	
		Pavement only	Total maintenance
Brick pavement			
Less than 500	31.35	$165	$414
500–1000	64.33	99	199
1000–2000	62.69	109	219
Over 2000	87.77	279	493
Total	246.14 Average	$174	$337
First-class concrete pavement 1:2:4 or better			
Less than 500	114.61	$ 62	$172
500–1000	158.28	54	152
1000–2000	199.46	76	226
Over 2000	98.28	149	402
Total	570.63 Average	$ 80	$230
Total of above types			
Less than 500	2384.17	$422	$543
500–1000	2181.23	438	595
1000–2000	1522.76	422	618
Over 2000	611.04	495	735
Total	6699.20 Average	$433	$595

Life of Pavements. Macadam Pavements.—Division 4 data on durability of these pavements are reliable. Table 101 gives the age at which actual reconstruction has occurred. This table is interesting but not conclusive, as the records apply to the weaker roads under exceptional traffic which tends to shorten the age reported. On the other hand, these roads were permitted to remain in poor condition for from 2 to 4 years beyond the time at which they actually needed resurfacing. These data give the average age for water-bound macadam as 8.9 years and bituminous macadam as 8.4 years, Topeka on concrete at 7 years, and second-class concrete (1:2½:5 mix) as 6.5 years. These results are inconclusive unless verified by supplementary data.

Table 102 gives the mileage of the different types which need resurfacing at their present reported age. These data are fairly reliable in determining range of life and probable maximum. According to these tables, supplemented by judgment in regard to modifying conditions, the reasonable life of the macadam base pavements is placed as follows: (Table 103 p. 534).

These ages are somewhat less than usually adopted, since they are based on a maximum permissible Vialog coefficient of roughness of 250′ per mile without excessive yearly maintenance expendtures (see pp. 545 to 552). This limit of roughness is somewhat

TABLE 102.—APPROXIMATE PERCENTAGE OF MILEAGE WHICH NEEDS RESURFACING AT DIFFERENT AGES

Present age in years, January, 1923	Water-bound macadam		Penetration bituminous macadam		Asphaltic concrete on macadam base		Asphaltic concrete, concrete base		Second-class concrete		Gravel	
	Total miles at each age (column 1)	% of column 1 which needs resurface	Miles	%	Miles	%	Miles	%	Miles	%	Miles	%
1	26.3	0	15.9	0	1.8	0	3.9	0				
2	8.9	0	3.1	0	0.7	0	2.3	0				
3	11.9	0	8.7	0	2.4	0	1.1	0				
4	27.2	11	23.4	0	0.5	0	....	.	0.3	0		
5	27.7	30	8.2	6	6.2	30	4.1	25	1.66	100	2.4	100
6	55.5	25	9.9	20	0.2	0	2.4	0				
7	51.8	50	11.0	35	2.1	0	0.7	0	8.38	100	2.9	0
8	33.4	60	15.5	24	...	..	1.4	0				
9	....	...	28.8	30	...	..	19.3[1]	75	6.36	100		
10	16.1	100	33.7	30	..	..	3.0	0	4.14	100		
11	13.0	100	48.0	68								
12	32.8	100	28.6	78	3.1	30						
13	33.1	100	3.4	50								
14	25.1	100										
15	17.7	100										
16	8.2	100										
17												
18												
19												
20	1.3	100										
21	1.3	100										

Modifying conditions and conclusions.	Average traffic Class II pavements rather weak considering soil and traffic. Range in durability, 4 to 10 years. Reasonable life for Class II for well-designed road probably 8 to 10 years.	Average traffic Class II pavements a little under adequate strength. Range in durability, 6 to 15 years. Reasonable life for Class II traffic well-designed pavements probably 10 to 12 years.	Average traffic Class I. *Data not conclusive.*	Poor construction. [1]Overburned mixture. *Data not conclusive.*	Class II traffic. Poor type of pavement. Reasonable life less than 5 years.	*Data not conclusive.*

less than has heretofore been customary, but it seems certain that on the main roads it is desirable to avoid the extreme roughness often tolerated for the last 3 to 5 years before reconstruction is commonly resorted to. The age, before reconstruction, can be increased by excessive maintenance expenditures, but this has little effect on the total cost of the combined items of maintenance and renewal; what is spent on one is saved on the other.

TABLE 101.—SHOWING AGE AT TIME OF ACTUAL RECONSTRUCTION

Age in years	Water-bound macadam			Bituminous macadam, penetration method			Bituminous macadam, mixing method, concrete base			Second-class concrete and Hassam concrete		
	Total resurfacing = 173 miles			Total resurfacing = 43.6 miles			Total resurfacing = 1.22 miles			Total resurfacing = 20.46 miles		
	Mileage rebuilt	Percentage of total	Accumulated percentage of total	Mileage rebuilt	Percentage of total	Accumulated percentage of total	Mileage rebuilt	Percentage of total	Accumulated percentage of total	Mileage rebuilt	Percentage of total	Accumulated percentage of total
3	0.30	0.2	0.2	.....	....	......	...	..		0.65	3.2	3.2
4	20.86	12.1	12.3	2.28	5.2	5.2	...	...	.....	4.29	21.0	24.2
5	5.94	3.4	15.7	0.77	1.8	7.0	..	..	.	3.53	17.2	41.4
6	16.13	9.2	24.9	5.13	11.8	18.8	0.40	32.8	32.8	1.14	5.6	47.0
7	13.80	8.0	32.9	8.81	20.2	39.0	....	..	.. ..	2.31	11.3	58.3
8	22.33	12.9	45.8	7.55	17.3	56.3	.	..	...	5.62	27.4	85.7
9	26.90	15.6	61.4	2.92	6.7	63.0	0.82	67.2	100.0	1.92	9.4	95.1
10	15.05	8.7	70.1	2.81	6.4	69.4	...	..	. ...	1.00	4.9	100.0
11	19.74	11.4	81.5	11.21	25.7	95.1						
12	4.44	2.6	84.1	2.12	4.9	100.0						
13	11.48	6.5	90.6									
14	9.46	5.5	96.1									
15	2.38	1.4	97.5									
16												
17	2.15	1.2	98.7									
18												
19	2.20	1.3	100.0									

TABLE 103.—TABLE OF ESTIMATED REASONABLE LIFE, FLEXIBLE-BASE PAVEMENTS

	Water-bound macadam	Bituminous macadam	Asphaltic concrete on macadam base
Class I traffic.......	Not suitable	6–12 years	8–14 years
Class II traffic.......	8–12 years	10–13 years	10–15 years
Class III traffic.......	10–13 years	12–15 years	Not advised
Class IV traffic[1].......	12–15 years	Not advised	Not advised

[1] Class IV is a very indefinite condition.

Table 97 uses the low limit. Under really good conditions of design and maintenance the upper limit can be used and even extended a few years.

Rigid-base Pavements.—The durability of rigid-base pavements on heavy-traffic rural highways cannot be very definitely settled as yet, as there are not enough miles of sufficient age to develop reliable data. Rural highway conditions are considerably different from city-street conditions due to concentration of traffic along an unprotected pavement edge and poor subsoil drainage systems on rural roads. City records are not really up to date on account of changed traffic conditions, so that the life assigned to rigid-base pavements is at best a careful guess which is controlled by the excellence of construction and maintenance practice for the district under consideration. The values assigned in Table 97 represent the best judgment of our local engineers reinforced by city records, and are for moderately good construction and maintenance conditions. The effect of good work on life is shown graphically on page 546. Supporting data are included below by a quotation from Blanchard's "Handbook."

TABLE 104.—CONSENSUS OF OPINION OF CANADIAN ENGINEERS AS TO DURABILITY (11)[1]

Pavement	Small repairs, years	Extensive repairs, years	Complete recon- struction, years
Asphalt block	5–10	10–12	12–15
Asphalt concrete	4– 6	6– 8[a]	8–10[a]
Bitulithic	5– 8	10–15	15–20
Brick	8–10	10–15	15–18
Cement concrete	5– 8	10–12	15–18[a]
Scoria block and stone	10–15	15–20	20–30
Sheet asphalt	4– 8	10–15	15–18
Untreated wood block	3– 5	6–10	10–12
Treated wood block	8–10	12–15	15–18[a]

[1] "American Highway Engineer's Handbook," John Wiley & Sons, Inc., New York.
[a] These pavements have not attained an age in Canada sufficient to place the figures beyond conjecture.

OPINIONS OF CANADIAN ENGINEERS AS TO NATURE OF DETERIORATION 11[1]

"*Asphalt block.*—Edge-wear forming holes and cobbles; worn pavement noisy and hard to keep clean; disintegration of bottom of blocks.
"*Asphaltic Concrete.*—Wears in holes; susceptible to marking in hot, sunny weather.
"*Bitulithic.*—Wears evenly, with a tendency to holes under heavier traffic susceptible to surface marks in hot weather.
"*Brick.*—Edge-wear and cobbling under heavy traffic; increased noise and unsanitariness as wear increases; when worn badly it slows down traffic.
"*Sheet-asphalt.*—Wears slowly into large holes or patches; heavy traffic causes ruts; cracks hasten deterioration.
"*Wood Block.*—Rots rapidly if untreated, retarding traffic; treated block wears slowly and evenly; frost intrusion causes buckling.
"*Stone and Scoria Block.*—Wears gradually into holes and cobbles; unsanitary when roughened.
"*Cement-concrete.*—Wear is slow and even, if well placed; cracking and chipping develop otherwise."

TABLE 105—*Continued*

Road No.	Type	Cost per mile, shoulders	Cost per mile, pavement	Cost per square yard, pavement	Remarks
61	Water-bound macadam—oiled	$ 25	$585	$0.062	Needs reconstruction
810	Water-bound macadam—oiled	198	308	0.033	Perfect condition
648	Water-bound macadam—oiled	94	284	0.030	Good condition
403	Water-bound macadam—oiled	52	434	0.053	Good condition
492	Water-bound macadam—oiled	35	250	0.027	Poor condition
5507	Water-bound macadam—oiled	71	379	0.040	Good condition
1290	Water-bound macadam—oiled	152	397	0.057	Fair condition
5021	Penetration bituminous macadam	139	335	0.035	Good condition
5046	Penetration bituminous macadam	198	350	0.037	Moderately good
62	Penetration bituminous macadam	55	227	0.024	Good condition
495	Penetration bituminous macadam	38	119	0.013	Good condition
5271	Penetration bituminous macadam	80	435	0.046	Fair condition
502	Penetration bituminous macadam	159	119	0.014	Good condition
80	Penetration bituminous macadam	27	164	0.018	Poor condition
63	Asphaltic concrete—macadam base	115	139	0.015	Good condition
253	Asphaltic concrete—macadam base	495	0	0.0	Excellent condition
5	Asphaltic concrete—macadam base	90	80	0.009	Good condition
5	Asphaltic concrete—cement-concrete base	152	127	0.013	Fair condition
538	Asphaltic concrete—cement-concrete base[a]	60	459	0.050	Fair condition
5302	Asphaltic concrete—cement-concrete base	366	172	0.018	Excellent condition
634	Asphaltic concrete—cement-concrete base	20	157	0.017	Poor condition
5279	Asphaltic concrete—cement-concrete base	26	183	0.019	Poor condition
1328	Asphaltic concrete—cement-concrete base	0	75	0.008	Good condition

5421	Cement concrete—1:2:4 mix or better	110	151	0.016	Excellent condition
5425	Cement concrete—1:2:4 mix or better	89	30	0.003	Fair condition
5423	Cement concrete—1:2:4 mix or better	128	8	0.001	Good condition
6	Cement concrete—1:2:4 mix or better	52	70	0.007	Good condition
5469	Cement concrete—1:2:4 mix or better	58	27	0.003	Good condition
1499*A*	Cement concrete—1:2:4 mix or better	22	10	0.001	Good condition
172	Brick on concrete base (cement-grouted joints)	112	56	0.006	Good condition
401	Brick on concrete base (cement-grouted joints)	56	56	0.006	Fair condition
6	Brick on concrete base (cement-grouted joints)	0 Village	131	0.013	Good condition
5435	Brick on concrete base (cement-grouted joints)	0 streets	79	0.008	Good condition
1430	Brick on concrete base (cement-grouted joints)	0 curbed	54	0.006	Good condition

Selected Typical Roads.—The second method of deriving normal costs is the selected-road method. Table 105 gives an idea of what can be done if the roads are well designed and maintained as compared with weak design and spasmodic maintenance.

Table 105 shows that firm water-bound macadam can be maintained under Class I traffic for about 6 cts. per square yard per year exclusive of renewals; under Class II traffic, for about 3.5 cts., but that weak design under even Class III traffic will raise the maintenance to about 6 cts.

Firm bituminous penetration can be maintained under Class I for 3.5 cts. for a long term of years (see road 5021); under Class II traffic for about 1.5 to 3 cts., but if the foundation is weak it jumps up to around 4.5 cts. (road 5271).

Asphaltic concretes on firm macadam base under Class IIA traffic have been maintained for a long term for from 0.9 to 1.5 cts. per square yard per year exclusive of renewals. The maintenance cost for these roads is about the same as for the same surface on a cement-concrete base.

The data on cement concrete and brick are not conclusive, because of the short age of pavements on which definite data exists in comparison with their probable life. During 1925 the maintenance on concrete roads rose from $80 to $230 per mile for pavement alone or 2.3 cts. per square yard average for the division as a whole.

Budget Estimates.—Budget estimates are desirable, as they insure a careful consideration of the needs of the situation and tend to produce a better-balanced ratio of expenditures over the territory as a whole. The following tabulation shows the schedules used in preparing budgets for Division 4. Patrol costs are figured by an actual itemized patrol layout, as shown in Fig. 178. Material and oiling estimates are based on careful field inspection of the conditions on each road modified by previous experience with increases due to winter wear. Equipment is carefully checked up and estimates of additions made. Salaries for supervision and office force can be closely approximated from past records. The total of these schedules becomes the budget estimate. These estimates are prepared for each county. The schedules are interchangeable in amounts as far as expenditure goes within the county, which gives the necessary flexibility, but funds allotted to one county cannot be transferred to any other county. This restriction gives the necessary stability. At one time an effort was made to hold strictly to expenditures for each schedule, such as patrol, materials, equipment, etc., but this was found to be impracticable and the present system was adopted.

TABLE OF MAINTENANCE FUNDS—DIVISION 4. TABLE SHOWING EXPENDITURES AND PERCENTAGE OF SAME TO TOTAL EXPENDITURES, EXCLUDING C AND E FUNDS

Expenditures include schedule appropriations, 10 per cent funds, general funds, and maintenance taxes. Federal aid funds excluded. Expenditures are for calendar year

	1920 (780 miles)*		1921 (840 miles)	
	Expenditures	Decimal part	Expenditures	Decimal part
Sch. A—Salaries	$ 40,573.05	0.1050	$ 49,059.32	0.1041
Sch. B—Office supplies	756.89	0.0020	657.19	0.0014
Sch. B—Livery	5,007.70	0.0130	5,613.31	0.0119
Sch. B—Travel	3,058.43	0.0079	2,756.52	0.0059
Sch. B—Communication	247.69	0.0006	247.06	0.0005
Sch. B—Office rent	125.00	0.0003	1,000.00	0.0021
Sch. B.—Advertising, printing	190.35	0.0005		
Sch. B—Purchase trucks, mixers, and other large equipment	1,624.74	0.0042	11,061.96	0.0235
Sch. B—Purchase war materials				
Sch. B—Purchase tools	6,011.62	0.0155	8,240.33	0.0175
Sch. B—Repairs to equipment	8,323.09	0.0215	11,890.84	0.0252
Sch. B—Insurance for equipment	3,480.18	0.0090	700.08	0.0015
Sch. B—Storehouse construction	6,416.01	0.0166	3,628.21	0.0077
Sch. B—Storage rental	1,955.12	0.0051	1,967.44	0.0042
Sch.				
Sch. C—Resurface	6,384.29	......	44,168.08	
Sch. D—Surface treatment	62,078.16	0.1606	116,420.25	0.2470
Sch. E—Reconstruction	481,013.03	......	358,530.44	
Sch. F—Foreman, labor, trucks, etc.	129,914.73	0.3361	155,883.86	0.3307
Sch. F—Rental equipment	1,007.50	0.0026	870.75	0.0018
Sch. F—Supplies for equipment	9,937.74	0.0257	8,284.47	0.0176
Sch. F—Materials	105,856.60	0.2738	93,046.04	0.1974
Sch. F—Drainage rights				
Total—excluding C and E funds	$386,564.53	1.0000	$471,327.63	1.0000
Total—including C and E funds	$873,961.85	......	$874,026.15	

$$\text{Decimal part} = \frac{\text{Schedule expenditure}}{\text{Total expenditure minus C and E.}}$$

To calculate proposed schedule appropriations for ensuing year multiply (total appropriation minus C and E) by average or proposed decimal part.

* NOTE.—For % of mileage of different types see Table 110, page 582.

Appropriations	1920	1921
Schedule appropriations (budget)	$755,000.00	$735,520.00
10 per cent funds—(used)	64,289.62	85,926.11
General funds—(interest, etc.)	14,682.54	16,892.69
Maintenance taxes—towns	33,950.00	34,600.00
Maintenance taxes—villages	8,318.50	8,479.35

TABLE 106.—NEW YORK STATE MAINTENANCE COSTS (1921)

Type of pavement	Miles maintained		Cost per mile		
	Including change to more durable type	Including change to more durable type	Excluding change to more durable type	Excluding change to more durable type	Pavement only, excluding change to more durable type
Bituminous macadam, penetration method, macadam base, asphalt binder	3201.87	3186.82	$1033	$ 770	$ 581
Tar binder	255.06	255.03	543	540	341
Concrete base, asphalt binder	45.21	45.21	560	560	446
Tar binder	16.81	16.81	363	363	250
Water-bound macadam	2599.04	2536.29	2007	951	813
First-class concrete, 1:2:4 mix or richer	730.45	730.45	225	225	81
Second-class concrete, 1:2½:5 mix or leaner	187.65	181.50	1957	874	710
Hassam concrete	49.03	49.03	679	679	395
Brick pavement	280.45	280.19	352	307	226
Gravel	141.14	136.30	1861	890	767
Bituminous macadam:					
Mixed method, Topeka concrete base	65.42	65.01	831	299	128
Topeka, macadam base	29.72	29.72	769	769	354
Amiesite, concrete base	6.25	6.25	440	440	215
Amiesite, macadam base	4.88	4.88	53	53	10
Open mixed, concrete base	17.09	17.09	413	413	166
Open mixed, macadam base	15.43	14.68	3869	462	288
Bitulithic, concrete base	15.12	15.12	244	244	231
Warrenite-bitulithic	1.81	1.81			
Willite, concrete base	1.66	1.66			
Bitoslag, concrete base	2.31	2.31	37	37	
Sheet asphalt, concrete base	0.61	0.61	229	229	229
Gravel mixed, gravel base	8.38	7.72	5154	1978	1757
Henderson, macadam base	0.51	0.51			
Block pavements:					
Asphalt, concrete base	18.34	18.34	347	317	214
Asphalt, macadam base	1.73	1.73	1271	1271	1082
Wood	0.26	0.26			
Stone	3.58	3.58	9	9	
Brick cubes, macadam base	0.32	0.32	104	104	104
Bituminous subbase	15.70	15.70	332	332	266
Bituminous macadam, three-layer method	7.29	7.29	734	734	448
Kentucky rock, asphalt	17.55	17.55	686	686	614
Rocmac	0.15	0.15			
Three-strip dual	6.81	6.81	134	134	120
Two-strip dual	33.99	31.69	1 562	332	146
Total	7781.62	7688.42			
Average cost per mile	....	....	$1280	$ 744	$ 582

Average Annual Maintenance Cost per Mile of Principal Types for the Past 7 Years, Excluding Change of Type

	1915	1916	1917	1918	1919	1920	1921	Average
Bituminous macadam, penetration method, asphalt and tar.........	$ 510	$ 483	$ 408	$ 557	$501	$590	$753	$543
Topeka on concrete base	...	205	245	435	144	335	299	277
Topeka on macadam base	....	256	393	1056	443	985	769	650
Water-bound macadam	1055	906	970	739	694	797	951	873
Brick...	190	176	222	251	247	242	307	234
First-class concrete..	129	141	112	160	214	227	225	173
Second-class concrete	1050	1080	1127	791	761	868	874	936
Gravel	955	587	918	909	771	704	890	819
Average all types.	$ 750	$ 651	$ 643	$ 608	$560	$631	$744	

Table 106*A*.—Supplementary Extensions and Author's Comments
New York maintenance costs

Type of pavement	Reported 7 year average per mile	Supplementary comments[2]				
		Approximate cost pavement only		Relation[1] of reported cost to normal cost under proper conditions	Recommended allowance	
		Per mile	Per square yard		Normal condition	Unusually favorable
Penetration bituminous macadam..........	$543	$400	$0.043	High	0.035	0.025
Topeka on concrete base......	277	150	0.016	Normal minus	0.020	0.015
Topeka on macadam base....	650	500	0.053	High	0.025	0.020
Water-bound macadam.......	873	720	0.077	High	0.070	0.060
Brick.......	234	100	0.011	Normal	0.015	0.010
Cement concrete (1:2:4 or better)............... ...	173	50	0.006	Very low	0.015	0.010
Cement concrete (1:2½:5 mix)	936	800	0.085	Normal	Poor	type
Gravel............... .. .	819	680	0.073	High	0.060	0.030

[1] Considers age, traffic and adequate design strength.
[2] Assuming pavement is used under proper volume of traffic (see page 6).

TABLE 107.—SUMMARY OF MAINTENANCE EXPENDITURES DEC. 1, 1921, to NOV. 30, 1922, STATE OF MASSACHUSETTS

Expenditures by types

Type	Cost	Miles	Average cost per mile	Square yards	Average cost per square yard
Granite block	$ 53.80	1.114	$ 48.294	20,204	$0.0026
Cement concrete	11,789.40	96.458	122.223	1,044,399	0.0113
Bituminous concrete (asphalt)	19,469.50	163.526	119.060	1,771,329	0.0110
Bituminous concrete (tar)	1,251.74	3.528	354.801	37,826	0.0331
Bituminous macadam (asphalt)	55,603.29	252.677	220.056	2,699,764	0.0206
Bituminous macadam (tar)	109,656.39	255.737	428.786	2,735,921	0.0401
Bituminous macadam (hot oil)	34,041.54	40.330	844.075	354,001	0.0961
Water-bound macadam	318,689.60	417.269	763.751	3,989,280	0.0799
Gravel	107,686.72	152.748	704.930	1,407,036	0.0765
Dirt road	22,641.50	12.760	1,774.412	137,052	0.1652
Brick	74.39	0.134	555.149	1,987	0.0374
Sand and clay (layer)	2,425.81	2.670	908.543	25,067	0.0968
Wood block	0.00	0.054	0.000	568	0.0000
Totals	$683,383.68	1,399.005	$ 488.478	14,224,434	$0.0480

Comparison modern and old types

	Modern types	Old types	Totals
Length (miles)	773,228	625,777	1,399,005
Area (square yards)	8,311,199	5,912,436	14,224,435
Per cent of area	58.4	41.5	100
Cost	$197,898.51	$485,485.17	$683,383.68
Per cent of cost	29.0	71.0	100
Cost per square yard	$0.0238	$0.0821	

Value of Smooth-riding Quality of Pavements.—A smooth easy-riding pavement is desirable on the score of personal riding comfort, reduced cost of motor operation (gasoline, tires, and jar damage), and reduced stress on the pavement due to impact of heavy trucks. Surface irregularities will vary with the type of pavement, care of construction, age of pavement, and perfection of maintenance, and they gradually increase as the pavements age until reconstruction becomes necessary.

Probably the most satisfactory method of measuring such irregularities at present (1923) is by means of the Vialog invented by Harley Dunbar of the New York State Department of Highways and manufactured by the Good Roads Machinery Co. of Kingston. This machine can be attached to an automobile. It records automatically in graphic form, as shown in the chart on page 548, the surface irregularities of the road. The machine is calibrated for the auto being used and the speed at which it is driven by means of careful test runs over a pavement of known irregularity. A duplicate record is made and the results agree remarkably well. The results can be examined for individual bumps or hollows or the average condition of the surface expressed as inches of irregularity per mile of road. This coefficient is reliable as a measure of relative roughness and of some value as a measure of absolute roughness. By means of records of this kind it is possible to set a reasonable limit of irregularity for acceptance of pavements of different types. It is also useful in keeping yearly records of the general condition of a state system and checks up the effectiveness of the maintenance and renewal programs. Being a mechanical device, it eliminates personal opinion and is a very useful instrument in highway work. The results of investigations of 2000 miles of New York State highways in 1923 can be summed up roughly as follows:

From the standpoint of riding comfort a pavement does not become disagreeably rough until the coefficient of roughness exceeds about 250″ per mile. The report on the 2000 miles investigated covers pavements of brick, asphaltic concretes, cement concrete, penetration bituminous macadam and water-bound macadam oiled. There is a wide range in roughness for each type, depending on age, maintenance, etc. All these types can be constructed originally and maintained for a considerable time in a comfortable condition for traffic. They all, in time, become rough and require resurfacing on the score of comfort and motor operating costs. The Vialog records verify the fact that the higher-priced pavements, such as asphaltic concretes, cement concrete, and brick, have a lower average roughness than the macadam type for the first few years, after which it becomes a matter of maintenance or reconstruction as to which has the advantage (see p. 546).

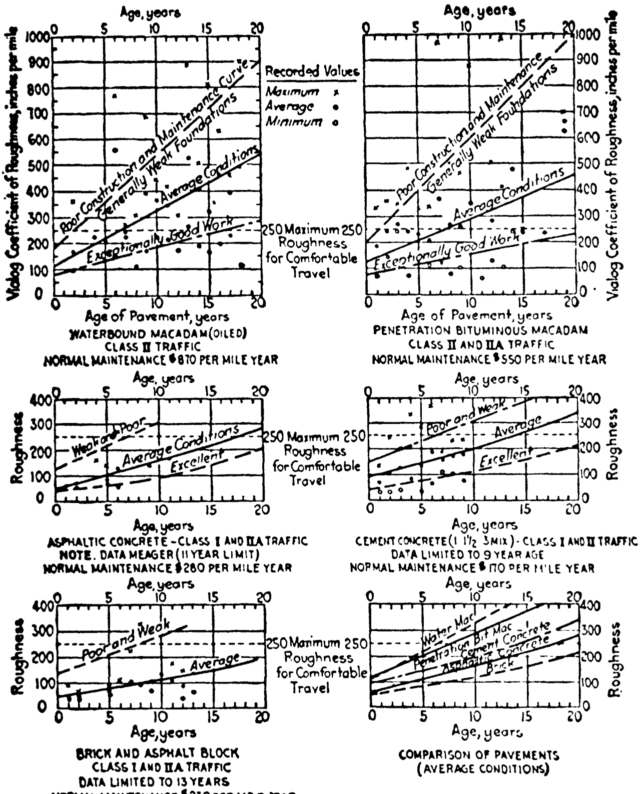


FIG. 175.—Graphs showing wide range and average value of Vialog coefficient of roughness for different pavements at different ages. (New York State 1923.)

Based on average maintenance, that is, no specially high maintenance has been used to hold the roughness to a specified maximum. Reconstruction is resorted to when normal maintenance becomes ineffective.

NOTE: These graphs show the wide range in results obtained under Public Works Programs and show conclusively the distinct advantage of good Engineering Control. Any type of pavement well designed, constructed and maintained will give satisfactory results as far as traffic operation cost is concerned.

To anyone personally familiar with this district these records have a human as well as a scientific interest. It is possible to trace the gradual loss or gain in interest in the success of the various types depending on the sympathies of the different administrations and they also show the effect of knowledge and experience of the inspectors with different types.

Mr. Dunbar gives some tentative values subject to future change as follows. He recommends a maximum value of 50″ for

Type of pavement	New pavements, good condition		Old pavements, poor condition
	Range, inches per mile	Average, inches per mile	Range, inches per mile
Cement concrete	25– 80	65	250– 600[a]
Brick	20– 80	65	250– 300
Asphaltic concrete	35– 90	75	250– 300
Penetration bituminous macadam	80–150	100	250– 800
Water-bound macadam (oiled)	90–170	110	250– 1000

[a] Low-grade 1 : 2½ : 5 concrete.

acceptance of newly constructed concrete, brick, or asphaltic con cretes and a maximum value of 100″ for new bituminous macadam. The average roughness of roads at present under reconstruction runs from 500 to 700″ per mile.

EFFECT OF ROUGHNESS ON MOTOR OPERATION COSTS

Roughness increases the gasoline consumption of motors and the tire and engine wear. Any attempt to figure this closely is a useless proceeding, but the general fact must be borne in mind in arriving at any reasonable conclusion as to the economic value of pavements and the necessity for effective maintenance. In time, Vialog records will be coordinated with gasoline consumption. No experiments along this line have yet been made. The following figures based on the present meager Vialog records must be considered as speculative as far as actual cost of operation is concerned. However, they check the gasoline consumption tests, and serve to illustrate the value of smoothness.

Mr. Dunbar states that on a road having a coefficient of about 50″, his chart distance measured by wheel revolutions was exactly equal to the actual lengths of the roads. Where the coefficients reached 500 to 700″, the chart distance by wheel revolutions exceeded the actual survey measured length of the road by from 100 to 200′ per mile. That is, roughness actually increases wheel revolution due to jump, free spin, and extra length, exactly how much is not known. A conservative assumption is 0.5% per 100″ increase in Vialog coefficient.

Roughness virtually increases the rate of grade of the road. A conservative figure is 0.2% per 100″ increase in Vialog coefficient.

Considering gasoline consumption alone and using the values given on page 98 of the first book of this series, "Highway Location," for distance and grade an increase in cost of motor operation of about 8% can be derived.[1] This amounts to about $100 per mile per year for 100 average vehicles daily (36,500 yearly) for each 100″ increase in Vialog coefficient. Making the same assumptions discussed in Highway Location in regard to the actual value of gasoline saving, considering pleasure traffic and snow and ice during a portion of the year, this is reduced to $50 per mile per year per 100

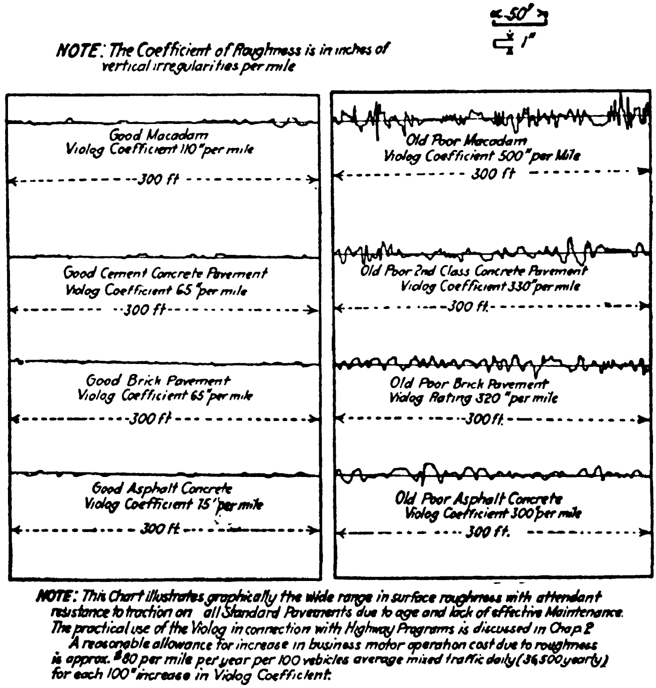


FIG. 176.—Vialog records of surface roughness (showing effect of age).

vehicles daily. For different volumes of traffic this is expressed in tabular form (p. 549) assuming a Vialog coefficient of 50″ per mile as perfection.

In a similar way an approximation can be derived for tire wear. Any actual cost value for tire wear due to minor differences of roughness is a very illusive quantity. Recent investigations have established that the difference in tire wear over gravel roads with loose small pebble surface as compared with a smooth new rigid

[1] Allowing for an average difference in Vialog coefficient of roughness of 50″ per mile between average macadam and average Class *A* surfaces, the difference in gas consumption is 4%. Actual experiment gives a value of 5%.

pavement may amount to as much as 1 ct. per vehicle mile. The actual difference in tire wear for ordinary road conditions, considering ice and snow in winter for any well-maintained gravel or macadam road with a bituminous surface carpet as compared with well-maintained rigid-base pavements, cannot amount to much. Assume some speculative figures merely to bring out the fact that there is some difference in cost of this phase of road travel over different types.

Ordinary tire wear on improved roads is assumed at 1.5 ct. per average vehicle mile for roughness of, say, 200″ Vialog rating. A well-maintained and renewed road system should not range more than 50 to 300″ per mile. The increase in wheel travel distance as

Volume of total traffic, vehicles per day	Vialog coefficient, inches per mile					
	50	100	200	300	400	500
	Values below are estimated yearly saving in business consumption of gasoline					
100	$0	$ 25	$ 75	$ 125	$ 175	$ 225
200	0	50	150	250	350	450
300	0	75	225	375	525	675
400	0	100	300	500	700	900
500	0	125	375	625	875	1,125
1,000	0	250	750	1,250	1,750	2,250
1,500	0	375	1,125	1,875	2,625	3,375
2,000	0	500	1,500	2,500	3,500	4,500
3,000	0	750	2,250	3,750	5,250	6,750
4,000	0	1,000	3,000	5,000	7,000	9,000
5,000	0	1,250	3,750	6,250	8,750	11,250
10,000	0	2,500	7,500	12,500	17,500	22,500

between 50 to 500″ rating amounts to about 2%. The added jar damage above 250″ rating probably does considerable more damage. Assume the same rate of tire cost as for gasoline previously derived, 8% per 100″ Vialog increase. This amounts to approximately $50 per mile per year for 100 average vehicles daily. Combining this value with the gasoline saving Table 108 is derived.

This table indicates in a general way the importance of keeping a system in fairly smooth-riding condition. To illustrate with the New York system, Division 4. The coefficient of roughness of most roads when actually reconstructed is about 500 to 700. This gives an average coefficient for the life of the pavement of about 300 to 400. The average volume of traffic for this territory is about 800 to 1000 vehicles per day (Class II traffic); that is, the cost of motor operation due to the degree of perfection actually attained probably is increased about $2000 per mile per year over a system in perfect condition. As later discussed, it is probably impracticable to attempt to maintain and renew a system with a maximum allowable roughness before renewal of under 250″ Vialog rating. If the system were maintained on this basis, the average coefficient would be about 150 to 200, which might result in a yearly saving in motor operation of about $800 per mile ($2000 minus $1200).

TABLE 108.—TABLE OF POSSIBLE DIFFERENCE IN MOTOR OPERATION COST PER MILE YEAR FOR DIFFERENT VOLUMES OF TRAFFIC AND DIFFERENT DEGREES OF SURFACE ROUGHNESS (1923 COST CONDITIONS)

(Based on gas consumption and tire wear)

Volume of traffic, vehicles per day	Vialog coefficient, inches per mile					
	50[a]	100	200	300	400	500
100	$0	$ 40	$ 120	$ 200	$ 280	$ 360
500	0	200	600	1,000	1,400	1,800
1,000	0	400	1,200	2,000	2,800	3,600
1,500	0	600	1,800	3,000	4,200	5,400
2,000	0	800	2,400	4,000	5,600	7,200
3,000	0	1,200	3,600	6,000	8,400	10,800
4,000	0	1,600	4,800	8,000	11,200	14,400
5,000	0	2,000	6,000	10,000	14,000	18,000
10,000	0	4,000	12,000	20,000	28,000	36,000

[a] Vialog rating of 50″ considered as practical perfection.

It is a well-recognized fact that funds have been insufficient and that the roads are too rough in their last stages before reconstruction. The average actual expenditure for maintenance and renewal has been about $1100 per mile for the last few years. The discussion of maintenance cost on page 524 indicates that approximately $1500 to $2000 per mile per year is necessary for this territory; the above data indicate that this increase is probably justified from the standpoint of economic motor operation as well as for the more obvious reason of comfort in riding.

Difference in Operation Cost on Different Surfaces.—A rough approximation can now be derived on the assumption that roads will be maintained and renewed on the basis of a maximum coefficient of roughness of 250″ Vialog rating.

Class B.—The macadam type of surface has an initial rating of 100″ and a final rating of 250″, or an average of 175″.

Class A.—The asphaltic concretes, cement concrete, and brick range from 50 to 250″, average 150″, giving an average difference between Classes *A* and *B* for total life of pavement of about 25 to 50″ Vialog rating. Expressed in money value as yearly cost and capitalized cost for different volumes of traffic, Table 109 is constructed.

In computing the total yearly cost of a pavement to the community it is just as well to give some weight to this item (see Official Reports, pp. 69 and 771).

For average conditions in Division 4, western New York, considering construction, maintenance, renewal, interest on construction, and difference in motor operation cost, the general conclusion to be drawn is that the macadam type of surface usually becomes uneconomical at a volume of traffic of between 1500 to 2000 vehicles daily. At this point an old macadam road can be resurfaced with asphaltic concrete and brought up to the operation efficiency of Class *A* pavements.

TABLE 109.—TABLE OF INCREASED COST OF OPERATION ON MACADAM TYPE OF SURFACE (CLASS *B*) COMPARED WITH CLASS *A* SURFACES (1923 COST CONDITIONS)

Volume of daily traffic	Increased yearly cost of operation		Capitalized value of increased cost at 5%	
	Per mile	Per square yard 18′ road	Per mile	Per square yard 18′ road
100	$ 20	....	$ 400	
500	100	0.01	2,000	$0.20
1,000	200	0.02	4,000	0.40
2,000	400	0.04	8,000	0.80
3,000	600	0.06	12,000	1.20
4,000	800	0.08	16,000	1.60
5,000	1,000	0.10	20,000	2.00

The student is again cautioned not to overemphasize the importance of hair-splitting economic value figures. They are included to illustrate general principles, and the values and conclusions given throughout this text have been adjusted to agree with actual observed road conditions.

Effect of Smoothness on Impact.—Table 67 (p. 372), gave a rough summary of the U. S. Office of Roads investigation on impact for known surface irregularities. Impact increases rapidly with the roughness and becomes a very destructive element unless the pavements are kept moderately smooth by effective maintenance. The section on Design (p. 373), assumed impact allowances based on ¼ to ½″ irregularities. An examination of the Vialog records show that such irregularities are quite common and continuous on the macadam type for an average coefficient of 120 to 200 per mile and occur at joints and cracks quite frequently on the rigid type of pavement with coefficients as low as 65 to 100 per mile. Where the coefficient rises to 300 to 500 per mile surface irregularities of 1 to 2″ are quite common and the pavement becomes not only uncomfortable and expensive to operate over but also is stressed beyond the design loading.

TENTATIVE MAXIMUM ALLOWABLE VIALOG RATING

From the standpoint of comfort, cheapness of operation, and cost of construction due to impact stresses it seems desirable to place a maximum limit of roughness at about 200 to 250 Vialog rating. As a road ages, the cost of maintenance will increase to retain this value and when the maintenance cost exceeds the proper limit for each type reconstruction or resurfacing must be resorted to. It is believed that if rural pavements are designed according to the principles outlined in Chapter VI, that the maintenance costs and average life before renewal become practical necessities, either on the score of economy or appearance or travel comfort will correspond closely with the figures given in Tables 97 and 105 pages 520 and 536.

An examination of the Vialog records in conjunction with ordinary New York State maintenance costs and procedure indicates that it is impracticable to attempt to reduce the maximum coefficient of allowable roughness below 200 to 250, as it would increase maintenance and renewal costs out of all proportion to the benefits derived.

A rough approximation of the maximum economic limit of maintenance cost per year for the different types is given below. When this limit is exceeded, resurfacing or reconstruction is desirable from the standpoint of paving cost.

Type of pavement	Class I traffic		Class II traffic		Class III traffic	
	Per square yard	Per mile	Per square yard	Per mile	Per square yard	Per mile
Asphaltic concretes	$0.17	$1700	$0.14	$1200		
Brick[a]	0.17	1700	0.14	1200		
Cement concrete[a]	0.17	1700	0.14	1200		
Bituminous macadam	0.17[a]	1700	0.15	1300	$0.12	$1000
Water-bound macadam (oiled)	....	.....	0.18	1600	0.13	1100

[a] Resurfaced with bituminous concrete.

As a matter of fact, maintenance costs rarely reach these maximums before renewal is resorted to, as the trouble and inconvenience to traffic from excessive maintenance work are the deciding factors rather than low total cost of maintenance and renewal. In Division 4, when it is necessary to spend about $1000 per mile to keep a road in fairly good shape, reconstruction is resorted to on the score of practical expediency.

The value of smoothness and the economic limit of maintenance expenditure before renewal is resorted to completes the discussion of maintenance economics.

MAINTENANCE METHODS

Methods will be treated in two parts: first, high-type, hard-surfaced pavement, and, second, low-type earth, sand-clay and gravel roads.

MAINTENANCE OF HARD-SURFACED PAVEMENTS

Organization.—A typical organization scheme for high-class state-improved systems with extensive mileage is shown in Fig. 177. For a system of this character (Division 4), the work can be effectively handled by a combination of patrol, gang work, and contract awards. The success of the program requires good judgment in

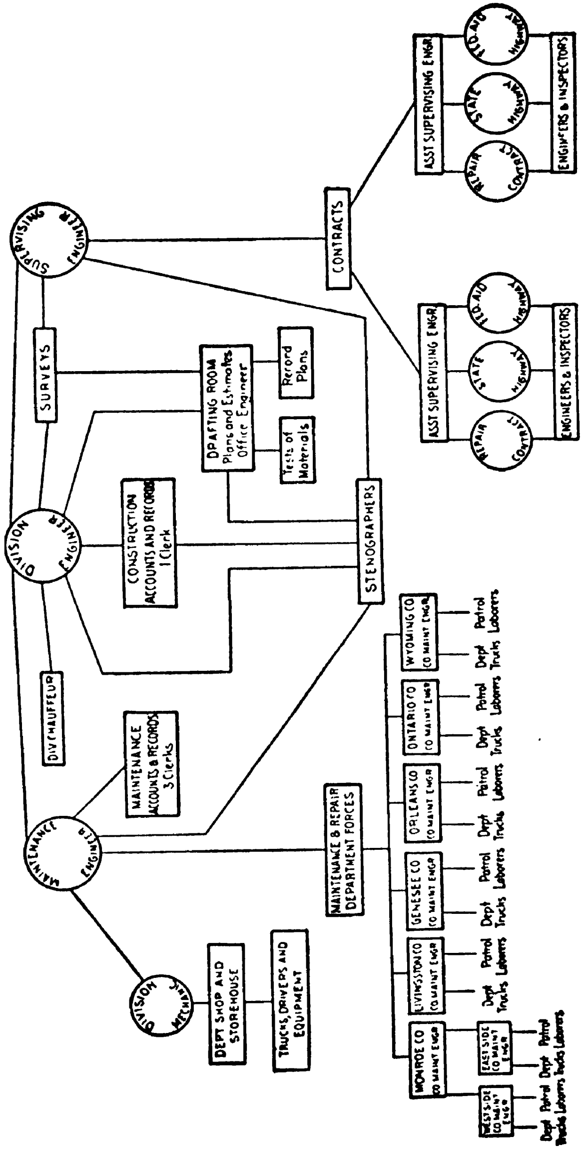


FIG. 177.—Organization chart, Div. No. 4, Western N. Y. 1000 miles state road under maintenance.

a well-balanced utilization of these different methods supplemented by correct use of materials and equipment.

a. One-man Patrol with Horse Rig.—This organization unit is very limited in usefulness for a high-grade compact system of heavy-traffic roads. It is useful for isolated roads under light traffic. The usual length of patrol is from 3 to 7 miles (see Figs. 178 and 179). The usual price paid for patrolman, including horse and rig, is about $4.50 per day (1922) for about 200 days per year. In Division 4, which is used for illustrative purposes, only 40 out of 860 miles were maintained by horse patrol in 1922. Patrol of this kind handles minor surface repairs of pavements.

On macadam roads the equipment used is as follows: one-half-cubic-yard wagon, pick, mattocks, shovel, broom, tamper, rake, asphalt bucket, scythe, straightedge, special equipment furnished from division storehouse for scrapping shoulders, or any special jobs; stone, bitumen, etc. delivered in stock piles by special truck gang.

On rigid pavements, a hook to clean out cracks and necessary chisels, bars, sledges, etc. for removing and replacing portions of the pavements are provided.

b. Small 1-ton Truck Patrol (One to Four Men).—This organization is a very useful and popular unit for maintenance work. It is flexible, speedy, and economical. The usual length of patrol ranges from 15 to 25 miles (see Figs. 178 and 179). The usual price paid for patrolman, including truck, is about $7.50 per day (1922). The usual price for extra helpers is from 35 to 50 cts. per hour. These patrols work about 200 days a year in northern climates. They take care of minor surface repair of pavements and keep shoulders, ditches, guard rail, culverts etc. in shape. On Division 4 this type of patrol takes care of 530 miles out of the total of 860 miles. On macadam roads, the equipment consists of 1-ton truck, shovels, picks, mattocks, push brooms, rakes, tampers, asphalt-pouring pots, ax, scythes, straightedge. These small tools are supplemented from the general division equipment by a small asphalt-heating kettle, and one-bag batch concrete mixer for cold patch work as required.

Where patrols of this kind work entirely on rigid-pavement roads, the force of men is generally reduced to two.

c. Three-ton Truck and Gang of Six to Twelve Men.—This organization unit is a recent development and serves very efficiently for work on roads requiring a large amount of surface patching, small areas of complete reconstruction, or extensive surface oiling. Gangs of this nature do all work not sufficiently important to warrant formal contract awards. They are, as a rule, responsible for the direct maintenance work on from 17 to 40 miles of road (see Figs. 178 and 179) and in addition deliver materials, reconstruct weak areas, and handle surface oiling. In Division 4 (used as illustration) 16 gangs of this nature handled the maintenance on 300 miles and in addition they reconstructed in 1922 approximately 4.3 miles of oad and handled all the surface-oiling treatment on the division.

The equipment required other than the small tools previously listed for patrol is roller, oil distributor, scrapers, tractors, any special machinery desired (division equipment list, p. 556).

d. Contract Work.—Extensive reconstruction and oiling are usually handled by contract awards, but oiling is gradually being taken over by these gangs, as they do better work, cheaper than the contract method.

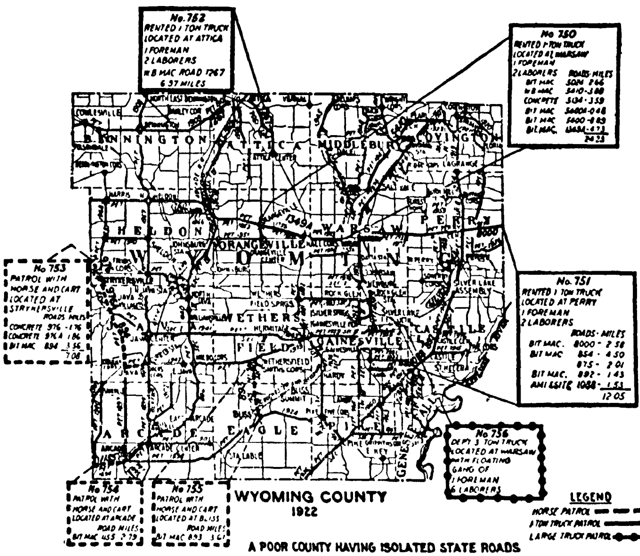


FIG. 178.—Maintenance patrol layout.

Large Equipment.—The number and kind of large machinery required will vary for each district. The following list, Division 4, gives an idea of the units found useful for a territory of this general character as described on page 523. A storehouse with repair and overhauling equipment is necessary.

Storehouse Organization (Division 4).—A workshop equipped with traveling hoists, electric drills, grinders, lathes, milling machine, drill presses, and small tools to enable mechanics properly to take down, overhaul, repair, and reassemble trucks and other equipment. Floor space of workshop should be large enough to place six or eight trucks at a time on the floor, say 40 by 80 or 50 by 100′.

In addition, there should be shed room enough to house all trucks during winter and equipment brought in for storage or overhauling, also a stockroom for small tools.

Mechanics at Storehouse.—One foreman mechanic is in charge of storehouse and all equipment, whether at storehouse or working on roads. He should have a small truck in order to make inspection and repairs to plant in field; also a helper to be at storehouse at all times. In winter months, in order to overhaul trucks and plant properly, there should be, in addition, at storehouse seven or eight mechanics. These latter could be the best among the truck drivers used in the summer on the road work.

LARGE MACHINERY EQUIPMENT (DIVISION 4)

(900 miles of state road)

10-ton rollers	7
5-ton caterpillar tractors	1
Steam scarifiers to attach to 10-ton rollers	2
Large blade scrapers (Austin Rip Snorter)	1
4000-lb. 10′ blade scraper	7
3-ton trucks, supplemented by a few additional rented trucks, 3 equipped with 800-gal, cold-oil distributors	20
¾-ton speed trucks	4
1-ton Ford trucks rented with patrolman	30
Asphalt distributors for penetration work with roller	3
Portable belt-conveyor unloaders	7
Street sweepers (rotary, horse drawn)	7
Small one-bag batch concrete mixers (cold patch)	15
Small concrete paving mixers (three-bag batch)	2
Necessary side forms and small tools	
Portable asphalt-heating kettles:	
300 gal	10
150 gal	12

For this division, quarrying and crushing equipment is not needed, as commercial stone can be obtained readily at a reasonable rate.

MATERIALS AND METHODS

The general principle of pavement maintenance is to prevent noticeable wear rather than to neglect minor flaws and then repair serious damage. Patch any hole large or small as soon as it appears. "Catch them young and treat them rough." Effective maintenance increases the life of pavements, increases traffic comfort, and reduces the cost of traffic operation. It is the best possible investment for the community and is essential to the success of any highway program. Maintenance includes the upkeep of ditches, shoulders, culverts, guard rails, guide signs, and pavement repair; the fundamental principle of pavement work is to repair like with like; that is, a hole in an earth road should be repaired with earth, gravel with gravel, macadam with macadam, Amiesite with Amiesite, Topeka with Topeka, brick with brick, etc. This seems perfectly obvious, but it is violated with clockwork regularity and the result is a patch that looks badly, wears badly, rides badly, and costs like the devil; it is a great temptation to use an easily manipulated patching material as a cure-all like David Harum's cure for spavin, ring-bone, pips in hens, etc., but it is poor policy. The following discussion outlines briefly the materials and methods in ordinary well-balanced practice.

Incidental Maintenance.—Shoulder maintenance consists in cutting weeds and grass either with farm mower or hand tools, maintaining the elevation and shape of the earth by means of blade scrapers hauled with trucks, roller or tractor (see Fig. 179), drags or hand-tool methods, or the addition of stone, gravel, etc. bound with bitumen on the main-traveled roads. The cost of this work has a wide range, depending on the excellence of the maintenance and on the width of pavement, and volume of traffic; for pavements having the widths recommended (p. 6), and where the maintenance is fairly effective, a fair set of costs is about as follows:

	Per year
Class I traffic (18′ pavements)..............	\$200–\$400 per mile
Class II and Class III traffic (16′ pavements)	\$100–\$200 per mile
Class IV traffic (12′ pavements)............	\$100 per mile

Ditch cleaning is largely a hand-tool proposition supplemented in some cases by blade-scraper work.

Guard rail, signs, etc. are routine mechanical operations.

This incidental work is about the same for all types of pavement.

Macadam Pavements.—Macadam-pavement maintenance involves repair of small areas of structural weakness, such as frost boils, etc., the repair of potholes in the surface, due to raveling of the top course, and the care of minor surface irregularities and roughness due to traffic wear.

Frost Boils.—Weak areas are dug up and replaced by new pavement of adequate depth reinforced by underdrains if necessary; this work follows the principle of depth design and construction methods described under Macadam Design and Construction (pp. 366 to 449). For a well-designed pavement, this kind of work rarely exceeds over 3/10 of 1% of the area per year. In 1917 in New York State 82 out of a total of 5600 miles broke through, or 1½% of the area. These breaks occurred largely on the old, thin roads, totaling 2000 miles, or a percentage of 4.2 for these old, weak pavements. For well-designed macadam pavements the item of frost-boil repair should not exceed \$50 per mile per year. Work of this kind is done by the large gang organization unit previously described.

Repair of Potholes.—Potholes due to ravel of the surface and not to structural weakness are of two classes: incipient depressions less than ½″ deep, and well-defined holes.

The first class of surface defect is remedied by sweeping out the hole, painting it with a thin coat of maintenance bitumen (cold-patch fluid) by means of pouring pot and broom, adding dustless stone or slag screenings, and tamping thoroughly. Screenings containing sharp flint or other hard material should not be permitted on account of tire damage.

Well-defined holes are repaired by cutting out the edges of the hole to a square edge, painting the edge and bottom with cold-patch liquid, and filling the hole with a cold-patch mixture of bitumen, stone, and sand, or stone and screenings. For depressions of about 1″ depth, ¾″ stone is used. Holes deeper than 1″ are dug out to the full depth of the top course, squared up and filled with regular-sized top stone ($1\frac{1}{2}$ to $2\frac{1}{2}''$), rolled with a 10-ton roller. Some engineers advocate light rolling, but experience indicates that the stone should be hard rolled but not crushed. Hot-binder-grade bitumen is then poured as for penetration macadam and filled with screenings. The patch is rolled lightly at once and thoroughly the next morning after the bitumen has hardened a little. A seal coat can be added, but is rarely necessary, as patching of holes is generally preliminary to extensive surface oiling. Work of this kind is done by the regular patrol gang. The amount of material required has a wide range for different roads.

In 1922, patching holes of this kind on Division 4 required from 10 to 60 tons of stone or slag per mile (50% $\frac{3}{4}''$ and 50% screenings) and from 200 to 1200 gal. of some kind of bitumen per mile. Cost of this work is difficult to segregate, but is probably about $150 to $200 per mile average on water-bound and about $50 to $100 per mile on bituminous macadam under Class II and Class III traffic.

Cold-patch bituminous compounds are a comparatively recent development. There are a number of satisfactory materials on the market, the best known being some form of

1. Asphaltic emulsion.
2. Tar cut back.
3. Asphalt cut back.

Good results can be obtained with all these materials. Locally, the best results have been obtained with the asphaltic emulsion, but it is the most expensive and will not keep over winter. The tar cut back is better for early spring use as it flows better at cool temperatures and is not damaged by winter storage. The cut back cold-patch mixture requires from 15 to 20 gal. of standard fluid per cubic yard of mixed stone and sand, or stone and screenings. Asphalt emulsion requires a little more, about 0.9 gal. per cubic foot of stone. The manipulation of this material is well described in the following quotation from the Barrett Company's "Vest Pocket Handbook." Maintenance men say that the greatest difficulty they experience in connection with the use of this material is the tendency on the part of the patrolman to use too much fluid as the mixture is easier to prepare under such treatment. Don't use too much bitumen, as it produces a patch which tends to creep and shove under traffic action.

MANIPULATION OF TARVIA-KP

"Tarvia-KP is a material that can be used for patching at any time of the year. It is as serviceable in winter as it is in summer.

"Intelligent use keeps bituminous macadam and other bituminous pavements in perfect condition all the time. Holes in wood block, brick, and stone block may be successfully patched with it.

"Tarvia-KP is used cold. It is used at any time, in any place. A batch of the mix may be made today, and need not be used until it is required, weeks hence. Batches may be made on rainy days and stored away for use later on, thus keeping the repair gang busy, rain or shine.

Holes and How to Patch Them

"Usually holes are one of two kinds: They either are breaks in the skin surface of the road, or they are deep depressions. The breaks can usually be mended by carefully sweeping them out and proceeding as is described below under the Penetration Method; the deep depressions are repaired by either the mixed method or the penetration method.

Deep Depressions. Mixed Method

"No hard-and-fast rule for proportioning the ingredients of the mix can be given, as sands and stone vary greatly in quality; but, in general, the following will be found to work satisfactorily:

"One cubic yard of a mixture of 1 part sand and 3 parts ¾" stone (stone held on ½" screen and passing 1¼" screen), 16 to 18 gal. of Tarvia-KP.

"These may be mixed by hand, using shovels and turning the mix about seven times, or they may be mixed in a concrete mixer of the batch type. A full minute is required to mix the materials in a mixer.

"After the mix has been made, it may either be used immediately on the road or it may be stored away for future use, covering it over with a tarpaulin or other suitable shield to keep the rain out. While the mix is set away it is curing. A hard crust will form on the outside, but this will in no way injure the bulk. The mix will be found to be in perfect condition 6 weeks, and even longer, after mixing. A pickax will loosen the tarred stone so that it may be shoveled into a cart or wheelbarrow and taken to the point of application.

FILLING HOLES

"Cut the sides of the hole vertically, and clean out the old material thoroughly. If the hole is over an inch in depth, fill to within 1" of the top with clean, broken stone, and tamp. The larger the stone up to 2½", that can be used for this work, the better. In other words, the stone is to furnish a firm foundation for the patch. The stone should be thoroughly rammed. Then paint the hole to be patched a light coat of Tarvia-KP (from ¼ to ½ gal. to the square yard), and deposit the mix therein. Place a quantity sufficient to fill the hole a little more than flush with the surface, and then tamp with a heavy iron tamper, or roll with a road roller until the surface of the patch is level with the surrounding surface of the pavement. Apply a light, seal coat of Tarvia-KP, cover with screenings, and the patch is ready for traffic.

"A dish-shaped shallow hole may be patched the same way, or by the penetration method.

"Great care must be taken to see that not too much Tarvia-KP is used. Use too little rather than too much. Too much acts as a lubricant for a time and tends to make the patch creep out of the hole.

"It does not matter how large or how small the patch is, Tarvia-KP will mend it.

PENETRATION METHOD

"Instead of mixing the ingredients, a penetration-method patch may be made. Owing to the fluidity of Tarvia-KP the method is not usually so good as the mixed method except for very small holes. The hole is cleaned out, a paint coat of the liquid, about ½ gal. to the square yard, is spread over the bottom, and 3" stone is deposited so that the hole is slightly more than filled. Tarvia-KP is then poured over the stone, using for each square yard about ½ gal. for each inch in depth of stone. The patch is covered with screenings and thoroughly tamped with a heavy iron tamper. A light seal coat about ½ gal. per square yard is then spread over the patch and it is allowed several days in which to set up before traffic is admitted, although this is not absolutely necessary.

PATCHING SHEET ASPHALT[1]

"The old sheet asphalt and binder should be removed and the hole cleaned out. Paint the bottom of the hole with a light coat of Tarvia-*KP*. Place the mix, roll, spread the seal coat of Tarvia-KP, and use screenings as before. The mix should be made as follows:

[1] For temporary relief only.

"One part sand, three parts stone, varying in size and grading of stone to suit the hole to be patched and local stone conditions. The mix should be so proportioned that it will tamp solid. No excess sand should be used or a rolling patch will develop. Sixteen to eighteen gallons of Tarvia-KP to the cubic yard is the usual requirement.

MISCELLANEOUS PATCHING[1]

"**Concrete Pavements.**—Holes in brick and cement pavements may be patched in the manner already described. Cracks in these pavements may be repaired by first cleaning them out thoroughly, and then filling them with a mastic made of Tarvia-KP and fine sand, using a slight deficiency of sand so that the liquid may find entrance into the crack or joint, and yet not thin enough to flow away easily.

"**Wood-block Pavements.**—It sometimes happens that a hole develops in a wood-block pavement and that there are no blocks available to patch with. A Tarvia-KP patch, made the same as for bituminous macadam, will serve well until such time as a wood-block patch can be substituted.

BRIDGE FLOORS

"The wear and tear on plank floors on bridges is considerable and has led a number of county engineers and others to take up the planks and substitute a concrete floor. This is always a dangerous change to make, for the bridges are not usually designed for the increased dead load of a concrete floor. A safer and better plan is to put a Tarvia-KP top over the plank floor. This will add comparatively little weight and will provide an excellent roadway. The method of construction is as follows:

"Sweep the planks thoroughly and when they are dry apply a light paint coat of Tarvia-KP, about ¼ gal. to the square yard. Place a 2" mixed top made of ¾" stone, sand, and the liquid in the proportions given under "Holes and How to Patch Them." This may be placed to have a depth of 2½" at the center and 1" at the sides, when rolled. Roll or tamp well, give a ⅓-gal. seal coat of the liquid and cover with screenings or sand.

RESURFACING OLD MACADAM WITH TARVIA-KP MIX[2]

"Usually the old macadam is scarified and reshaped with the addition of new stone if necessary. If, however, the surface is in fairly good condition, the ruts and depressions are repaired with a mix of crushed stone and Tarvia-KP. The road is then swept and a paint coat of the liquid applied to the entire road surface. On this a mix of sand, broken stone, and Tarvia-KP is spread to a depth of 2½" and rolled. When the rolling is completed, a seal coat of Tarvia-KP is applied and the new top is covered with pea gravel or stone chips. The road is allowed to set up for several days before traffic is turned onto it.

"The mix is made in a concrete mixer. For a small mixer a good proportion has been found to be 1 cu. ft. of sand, 4 cu. ft. of stone, and 3 gal. of Tarvia-KP. These are mixed for one full minute at least, and are then dumped in a pile to cure for a week. At the close of the day's work, a quart of kerosene and two shovelfuls of stone are revolved in the mixer to clean it out.

"If the mix shows a tendency to wave under the roller to any great extent, let it cure for a day or so. The volatile oils will evaporate, leaving the mix stiffer and firmer.

"Tarvia-KP is supplied in barrels. It can be stored without deterioration indefinitely, even in winter weather. It is well, however, to stir the contents before using. The barrels hold approximately 50 gal. Where there are facilities for storing the liquid in bulk, it may be supplied in tank cars. The storage tank should be supplied with coils or other means for heating the liquid in cold weather. Tank cars hold from 8000 to 10,000 gal. Tarvia-KP should never be heated in open tanks or kettles. It can never be mixed with very hot stone or sand.

"Use Tarvia-KP sparingly. You can always add, but you cannot subtract."

[1] For temporary relief only.

[2] Resurfacing with cold patch is rarely desirable.

Calcium Chloride Treatment.—Calcium chloride is often used as a temporary maintenance method for the first season on water-bound roads. About 1¼ lbs. per sq. yd. is applied by an agricultural drill for the first application and about 1 lb. per sq. yd. for succeeding applications. Two or three applications per season should keep a new Waterbound Road in good condition under Class III Traffic. The particular value of calcium chloride is to prevent ravelling the first season. Oiling is generally resorted to the second season after the road has been well compacted by traffic. For the cost of apply calcium chloride see page 1220.

Surface Oiling.—Surface oiling produces the final smooth, dust-less surface demanded by modern traffic. It is the final stage of macadam maintenance. All structural defects and surface holes must be repaired before surface oiling is done. There are three well-defined conditions in connection with this work which require different methods of treatment.

1. Light oiling to liven up a comparatively smooth surface.
2. Medium treatment to smooth and seal a rough surface.
3. Heavy hot oiling with thick cover mat to repair temporarily a road in very poor condition.

The light type of treatment is used where the surface is smooth but requires some bitumen to liven up the binder on penetration bituminous macadams. For a treatment of this kind, 0.2 to 0.25 gal. of bitumen is applied after the road is swept clean of dust and loose material of all kinds. The oil is applied by mechanical distributors, one-half of the road being treated at a time to avoid inconvenience to traffic. The oil is immediately covered with a light coat of clean, sharp sand, dustless stone, or slag screenings, using about 18 to 30 tons of cover per mile of 16′ road. The road surface must be clean when the oil is applied but slight dampness does no harm so long as water is not standing in pools. Either a light cold tar or light asphaltic oil can be successfully used. Locally, the tar is preferred for treating water-bound macadam or tar-penetration macadams, and the light asphaltic oil is preferred for treating asphalt penetration macadam (see Specification, items B. and R.C.O., p. 1397 and 1396). Sharp sand makes the best cover material, as it is less likely to cause excessive tire wear.

The medium type of treatment is used where the surface is rough. The manipulation is the same as for the light treatment except that a little more oil is used, from 0.3 to 0.4 gal. per square yard, and more cover, 80 to 100 tons per mile of 16′ road; ¾″ stones rolled until they are completely crushed may be used for cover, but the dustless screening cover rolled in is better practice.

The heavy, hot-oil treatment is used sparingly, as when a road gets into the condition where this method is needed it is generally better policy to do a good job of resurfacing. This method is used as a stop gap to save a road where the funds are insufficient for a new top course. From 0.6 to 0.7 gal. per square yard of binder-grade bitumen (either tar or asphalt) is applied hot to the well-cleaned surface and covered with from 100 to 130 tons per mile (16′ road) of a mixture of about 75% ¾′ stone and 25% dustless screenings.

This cover is rolled hard until all the $\frac{3}{4}''$ stones are crushed down to the screening size. The cost of this treatment ranges from \$1200 to \$1600 per mile of 16′ road (1922 cost conditions).

Surface oiling of either the light or medium method is routine procedure in macadam maintenance. It starts the second season on water-bound roads and continues at irregular intervals throughout the life of the surface. It costs on the average about \$350 to \$450 per mile for each oiling, which amounts to about \$300 per mile per year average under Class II traffic for the water-bound type. On penetration-bituminous-macadam roads it starts the second season on tar penetration and the third or fourth season on asphalt penetration and continues at irregular intervals as required. It costs about \$350 to \$450 per mile for each oiling, which amounts to about \$150 to \$200 per mile per year average under Class II traffic. All costs are 1922 cost conditions.

The following tabulation gives a rough idea of the relation of the different items of maintenance to total yearly cost for 16′ macadam roads well designed and reasonably well maintained.

Class of work	Bituminous macadam, Class II traffic, cost per mile	Water-bound macadam, Class III traffic, cost per mile
Repair of weak areas	\$50	\$50
Surface patching	\$ 50–\$100	\$150–\$200
Surface oiling	150– 200	200– 300
Shoulders, ditches, etc	150	100
Totals	\$400–\$500	\$500–\$650

NOTE.—Each of these items includes an allowance of 10% for administrative overhead and equipment.

The following quotation from the report of Fred Sarr, former Maintenance Deputy of New York State, reinforces the data given.

"Efficient maintenance of macadam pavements, particularly of the water-bound type, of which there are 2535 miles in the state system of improved highways, necessitates frequent surface treatments with bituminous material or constant patching of the holes that rapidly develop under the present-day motor-vehicle traffic.

"Frequent surface treatments are objectionable not only from a traffic standpoint, but from the fact that such treatments tend to build up an unstable mat of bituminous material and mineral aggregate on the surface of the pavement that is displaced by the fast-moving motor vehicle traffic, and develops a rough and uneven surface.

"It has, accordingly, been the policy of this Bureau to restrict the use of surface treatments and wear the surface mat down, as this is possible before giving another general surface treatment.

"This method, while tending to provide a smoother surface, requires constant patching during the later stages of the wearing-down process.

"Much time and thought have been given to the study of the results obtained by various methods of manipulation and materials used in patching macadam surfaces.

"In making these patches to pavements carrying any considerable amount of motor-vehicle traffic, it is necessary to bind the mineral aggregate with some form of bituminous material.

"Light asphaltic oils and refined tar products, similar to those used for surface treatments, have been used extensively for light, thin patches, painting the area to be patched with the bituminous material and covering with stone chips or sand.

"Heavy binders that require heating have been used in the same manner.

"The most satisfactory results have been obtained, where the required patch must be ½″ or more in depth, by mixing the mineral aggregate with a heavy asphalt or tar binder, cut back with light voltaic oils to a consistency that will mix readily with the mineral aggregate when cold, also with an emulsified asphalt binder in the same manner.

"The bituminous material and stone aggregate, being mixed either by hand or in a small concrete mixer, permits of a proper proportioning of the materials, which has been demonstrated to be about 6% in weight of solid bitumen or mineral aggregate used, or about 1 gal. of the cut back or emulsion per cubic foot of crushed stone.

"With asphalt cut back, the best results have been obtained by using a material made from an asphalt binder, having a penetration of about 165, cut back with about 33% in weight of naphtha.

"With tar cut back, the best results have been obtained with a material made from a refined tar binder having a melting point of about 60°C., cut back with about 40% in weight of tar oils, of which at least 60% shall distil up to 235°C.

"A very satisfactory material for patching purposes is an emulsified asphalt containing about 65% of asphalt binder having a penetration of about 165.

"This material may be diluted with water if desired, and may be mixed with wet mineral aggregate when found in that condition. It readily separates from the emulsified state when combined with crushed stone in the so-called open mix.

"The resultant adhesive qualities of an emulsified asphalt appear to be better than can be obtained by the same asphalt in any other form.

"The only tangible reason advanced for this result is that water in the emulsion may carry the binder into the pores of the material or pavement to which it is applied.

"The patch made with emulsified asphalt hardens to a condition of stability much quicker than one made with cold oils or tars or cut-back binder that we have used, and is, for this reason, preferable to those materials for patching work on heavy-traffic highways.

"Very good results have been obtained with the cut-back tar cold-patch material, particularly on medium- to light-traffic highways, where the patching material is not thrown about by traffic to any great extent.

"In order to obtain efficient results in patching with a tar binder, it is necessary to make a so-called close mix, by using a graded mineral aggregate having a minimum amount of voids, which, however, will not permit the volatile oils to evaporate as fast, and the patch to become stable as quickly as may be obtained with asphalt emulsion when used in the open mix. It is, accordingly, preferable when using tar to mix same with the mineral aggregate and leave in shallow piles for about 2 days before applying to the road surface.

"The necessity for using the close mix with tar binders is due to the fact that tar products are more susceptible to the heat and cold than asphalts.

"In other words, if starting with the two materials of the same consistency at 60°F. the temperature is raised to that of a pavement on a hot summer day, say 130°F., the tar is much more fluid than the asphalt and tends to flow away from the open minimum aggregate, and the open patch will show a tendency to ravel. Again, when the temperature is reduced to that of a pavement on a winter day, the tar becomes much more brittle than the asphalts and again the open patch with tar binder is much more liable to ravel out than one made with asphalt binder.

"A comparison of the result obtained with the two materials, each of which contains a quantity of the semivolatile oils sufficient to permit them to be applied to the surface of the pavement at 60°F. as a surface treatment, demonstrates that the tar, by reason of its greater fluidity on a hot summer day, will penetrate the old pavement to a greater extent than the asphalts, and thereby serves more as a binder to the old pavement. It is for this reason that cold tars are generally used as the first and second treatment of a water-bound macadam pavement. Subsequent treatments of heavy asphaltic oils, carrying about 65% of solid bitumen, will give more efficient and lasting results if used conservatively, that is, if the successive treatments do not follow each other too closely.

"When successive treatments are given every year as a dust layer to obviate the necessity for patching, cold tar is preferable in that it does not build up a mat on the surface of the pavement to the extent obtained with asphaltic oils.

"Provided a mat is built up with successive tar treatments, the same will generally lie flat and not shove under traffic, and develop a wavy and corrugated surface as is often obtained with too frequent surface treatments with asphaltic oils.

"This resulting difference is due to the aforesaid difference in consistency of the pavements at summer temperatures.

"The tar, being so fluid at summer temperature, appears to retain a smooth surface by the effect of gravity, while the asphalt simply softens sufficiently to permit the surface mat to be displaced by traffic, which displacement increases from day to day and often necessitates the entire removal of the old mat.

"Another factor to be considered in deciding upon the material to be used for the surface treatment is the condition of the old pavement.

"Where the old macadam is composed largely of small particles of crushed rock and dust, and is in a more or less loosened condition, and is subject to displacement by traffic, a light asphaltic oil is preferable to cold tar for surface treatments. The asphaltic-oil treatment develops into a mat or carpet over the macadam which remains more or less plastic, even at low temperatures, and displacement of the macadam under traffic does not result in the shattering and the ultimate destruction of the mat to the extent obtained under similar conditions with tar treatments.

"Also for the same reason, asphaltic oils give the best results on pavements where steel-shod traffic predominates.

"The best results obtained with tar treatments are where the old macadam pavement is clean or free from dust and where the pavement is firm and sound, and the stone fragments do not displace under traffic, and where motor-vehicle traffic predominates, also where a minimum amount of cover material is used in conjunction with bituminous material.

"Macadam pavements surface treated with tar are, however, much more slippery for horse traffic in cold weather than those treated with asphaltic oils.

"In my report of a year ago, I discussed to some length the subject of the extensive breaking through of many of the pavements during the spring months.

"Referring to such report, it will be noted that the total area actually broken through during the spring of 1916 was equivalent to 82 miles of pavement 16′ wide, and that the broken areas were distributed over many projects aggregating to a total of 1939 miles, of which an average of 4.2 % was broken through.

"During the season of 1916, about 75% of the total broken areas was substantially repaired, and about 238 miles of the weaker pavements were resurfaced.

"The spring of 1917 appeared to be a repetition of the previous year as to the amount of broken pavements.

"The result of a survey to determine the extent of the broken pavements, when tabulated, indicates that the total broken areas were, however, but 60 % of the total of the previous year.

"The total length of the various projects involved aggregated 2090 miles, about 9% larger than those reported in the previous year. Of this total length the equivalent of about 48 miles of pavement 16′ wide was broken up, or about 2½ % of the total length involved."

Asphalt, Topeka Mix, Amiesite, Etc.—The holes which develop in the bituminous-mixing-method-type wearing surfaces should be repaired as follows: Excavate the old material at the defective spot the entire depth of the course, so that the edges will present clean, vertical surfaces, these surfaces and the exposed foundation to be swabbed or painted with hot asphaltic cement or paving pitch, the hole then to be filled with a mixture similar to that used in original construction whenever practicable, using a sufficient quantity so that after consolidation or rolling (or tamping in case the extent of repairs is limited) the surface of the new patch will be ¼″ above the adjacent pavement. In case no local mixing plant is available, or the limited extent of repair does not justify the expense of treatment as above, holes may be repaired with the mixture of crushed stone and cold-patch asphaltic emulsion, as outlined for macadam surfaces page 558.

MAINTENANCE (CONCRETE PAVEMENTS)[1]

"Maintenance should be systematic and imperfections given immediate attention.

"**Cracks and Joints.**—The crack or joint should first be cleaned with a sharp-hooked pick and stiff brush, care being taken to remove all loose particles. If the opening is too narrow to permit cleaning in this manner, it should be cleaned with an air jet from an automobile tire pump. After thorough cleaning, tar should be poured into the crack in sufficient quantity just to flush over the edges. The tar should then be covered with coarse, dry sand. It will usually, be found desirable to remove old joint material to a depth of ¼ to ½″ below the surface of the concrete, so that new repair materials will have a good bond.

"Care shall be taken not to pour the tar in such quantities that an unsightly wide strip is made on the surface. By bending the spout of the tar pot so that a long, narrow opening is provided, the tar can be poured in a very thin, narrow stream. It needs but a little attention to pour cracks so that a narrow ribbon of tar 1 to 2″ in width is left.

"**Tar.**—Refined coal tar should be used, having a melting point (½″ cube method in water) of about 100°F. The tar should be heated from 225 to 250°F. at the time of application and may be applied by means of a sprinkling can with spray nozzle removed."

"**Surface Pockets.**—A pocket in the surface, 1 to 3″ in size, due to soft aggregates or the disintegregation of a lump of clay, or a piece of coal or wood, should first be thoroughly cleaned. It should then be filled with hard, small aggregates, ranging in size from ¼ to ⅜″. Hot tar should then be poured into the opening until the tar is flush with the surface. Very coarse, dry sand should then be sprinkled on the tar immediately.

"A cold mix of small stone and emulsified asphalt has been successfully used for this type of repair work (see p. 558).

"**Slight Depressions.**—If for any cause a slight depression has formed, it can be coated with tar, very coarse sand added, covered with tar and the whole covered with sand and tamped into place.

"**Replacements.**—If it is necessary to cut a hole through the entire thickness of the concrete slab, gravel should be placed in the subbase and thoroughly rammed, so as to form a compacted base on which the new concrete will rest. Where water has been allowed to stand in such a place, it should be compacted after the water has been removed and just before laying the concrete. Before placing the concrete the sides of the opening should be painted with a mixture of neat cement and water.

"The consistency of the concrete should be sufficiently stiff to require considerable tamping to bring water to the surface so that it may be possible to ram it thoroughly into place. A small tamp, made of a 2 by 1″ stick, 18″ long, should be used to ram the concrete adjacent to the edges.

"After the concrete is rammed into place, it should be struck true to the surrounding surface. A repair made in this manner cannot be told from the old pavement after a few months use.

"A new patch should be kept moist for at least 4 or 5 days, and protected from traffic at least 10 days. By the use of Lumnite cement patches, traffic can operate within 24 hours."

Repair of Concrete Pavement with Quick-setting Cement.—During the season of 1925, the New York State Bureau of Highways under the supervision of County Assistant Engineer J. A. Small repaired the cement-concrete pavement of Highways 5422 and 5469 in Orleans County, by removing broken pavement and replacing with concrete of Lumnite cement. The total area replaced amounted to about 3000 sq. yd. On Highway 5422, 6% of the total area was replaced, and on Highway 5469 2% of the total

[1] Quoted from Portland Cement Association Handbook.

area. These roads were built in 1914 of 1; 1½ : 3 concrete and were 6″ thick, not reinforced. The areas removed varied from corner breaks of about 1 sq. yd. to patches extending the full width of pavement, 90′ in length.

The defective pavement on one-half the pavement was repaired before doing anything with the opposite side. Traffic was not held up or inconvenienced to any great extent during the progress of the work except at the site of the work, where one-way traffic only was possible. The total length of pavement under repair at one time was not more than ½ mile, generally less. Traffic was warned by portable signs placed at each end of the work and moved as the work progressed. These signs were of wood, 5 by 7′ and lettered as follows:

DANGER

REPAIR WORK AHEAD

ONE-WAY TRAFFIC

GO SLOWLY

When traffic was heaviest flagmen were placed at each end of the location of the work to direct traffic. The average traffic was about 150 vehicles per hour. As no serious and very few minor accidents occurred either to vehicles or to workmen, the method of carrying on the work was considered satisfactory.

The first operation of the work was the stocking of repair materials, sand and stone, at convenient places along the road shoulder. These stock piles were placed so that a considerable amount of repairing could be done with one set up of the mixer. These piles were placed not nearer the edge of pavement than 3′ in order to lessen danger to traffic. Cement was kept at a central storehouse and brought on the job daily as used.

The next operation was the removal of defective pavement. Only those blocks that were shattered and uneven, and corner breaks that were depressed below the level of the adjacent pavement, were removed. Blocks or areas that were cracked but not out of grade with adjacent pavement were not removed, as it was soon discovered that no bond could be made between the new and the old pavement and therefore there was no value or use in replacing old pavement which was cracked only with a new patch that would also show the joint between new and old

The defective areas were broken up into one-man size pieces by means of air drills operated by an air compressor mounted on a four-wheel army auto trailer. The air compressor used on this job was an Ingersoll-Rand outfit, which operated two drills at the same time. The drill itself was a pavement breaker and was operated by the systems of splitting or wedging of a small piece of concrete at a corner or edge and then working back into the slab by breaking off successive small pieces. The gang consisted of two drill runners and two laborers. These laborers assisted in moving the machine and attended to spreading apart the broken pieces of concrete so that the next piece could be split off by the drill. It general, the drill was some distance ahead of the concrete mixing gang so that the concrete in the areas broken up by the drill was left in place so that traffic could go over them until the gang engaged in removing this concrete came along.

This gang removed the broken concrete, trimmed off the edge of the pavement, and excavated the subgrade to a depth of from 7½ to 9″ below the top of the adjacent pavement. Usually it was impracticable to load the broken material on trucks at the time of removal on account of traffic using the other side of the highway. Consequently, this material was piled along the shoulder for removal later, or used to widen out the shoulder at the location of the patch if practicable.

In the beginning of the work no care was taken in trimming the edge of the old pavement after the drilling operation, as it was not thought necessary. It was soon discovered, however, that if any of the new concrete extended over the old pavement in a comparatively thin layer, it invariably cracked or spalled off, leaving a ragged edge to the patch. Consequently, thereafter the raw edge of the old pavement was trimmed by means of a mason's spalling hammer to a line having no sharp corners, no top spalls, and with the edge as near vertical as possible or slightly undercut.

No attempt was made to strengthen the subgrade under the patches of small area for the purpose of preventing them from being depressed under traffic. It was considered that to make the foundations of the patch stronger than the adjacent old pavement would tend to allow the old pavement to be depressed below the new. Any defect in the foundations, however, such as large boulders found under the old pavement, were removed and the subgrade made as nearly uniform as possible. In a few cases, also, a foundation filler of 4″ of stone screenings was placed under some small patches to determine if such treatment would have any effect in sustaining the new patch.

The location of open holes of small size in the pavement was indicated to the traveling public by means of broken pieces of concrete placed at the ends and side towards the center of the highway. Large repair sections were barricaded with lumber barricades. Also at both ends of the section upon which work was being carried on were placed barricades with warning signs. All openings or fresh-placed patches were indicated by red lights at night.

The operation of filling the open holes with quick-setting concrete was the next. On a large area to be filled, the mixer was set as near the opening as convenient, when the holes were smaller, but comparatively near together, one set-up was used for holes within 100′ or so of each side of the mixer. The mixed concrete was taken from the mixer to place by means of wheelbarrows. When the holes were farther apart and scattering, the concrete was loaded into a ½-yd. dump-cart trailer and hauled to place by means of a speed truck. The mixer gang consisted of one man operating the mixer, four men shoveling into the mixer, and three or four men wheeling. The mixture used approximated 1:2:4 mix. Stone was of the ¾″ or No. 2 size, New York State Standard. Sand was local and generally ran coarse. Water was hauled to the location of the mixer in a tank truck trailer. This tank trailer was set up at the division shop and consisted of two 250-gal. army gasoline tanks mounted on a four-wheel army trailer. The mixer used was a one-bag Little Wonder with hoist.

In placing the concrete, a form was placed along the outer edge of the pavement and held in place by iron pins. The bottom of the excavations and edges of the old pavement were sprinkled with water. At first the edges of the old pavement against which the new concrete was to be placed were painted with a neat cement grout in hopes it would assist in obtaining a bond between the new and old. It was soon evident that no bond could be obtained strong enough to sustain the weight of traffic passing over the highway, or even to prevent cracks forming when the concrete contracted upon setting. Therefore no attempt was made to obtain any bond. Each separate patch must sustain the weight coming upon it. In the case of two adjacent corner breaks it was found that no advantage was obtained in replacing the expansion-joint material between the two new corners. In fact, it was better construction to make the two patches continuous, thus giving a larger bearing surface on the subgrade. Care was taken to get mortar next to the old concrete and to spade the concrete under the old slab where necessary. The new patch was screeded so as to give a smooth-riding surface in connection with the adjacent pavement. This made it necessary to place some patches across the pavement and some others longitudinally.

In curing the quick-setting Lumnite cement it was necessary to keep the concrete wet during the period of setting, at which time considerable heat is generated. Generally, within 5 hr. of mixing watering must be commenced, and the concrete kept wet or damp for about 15 or 20 hr. thereafter. This was done by means of a hand sprinkling pot. Water was kept in barrels at convenient intervals along the work. This sprinkling was done at night by the night watchman. Some difficulty was experienced in getting a man who would faithfully keep the pavement sprinkled. This man was on duty from 5 p.m. until 7:30 a.m. at which time the regular gang took up the work continuing sprinkling as long as the concrete showed any heat. Generally, sprinkling every 15 min. for the period of from 5 to 10 hr., and at half hours or longer intervals thereafter, was sufficient.

On the small patches where the subgrade had become wet and softened by water from sprinkling, traffic was kept off until the end of the second day or at a time of from 24 to 30 hr. after the concrete was mixed. Side forms were removed and the opening backfilled with some non-porous material as soon as possible after the patch was cured to prevent surface water from standing along the edge and softening the subgrade. These precautions were necessary to avoid, if possible, the settling of the patch by the weight of heavy vehicles going over it. As the barricades and obstructions of the previous day's work were removed at the end of the next, or second day, the work as stated above did not extend more than ½ mile, where traffic was inconvenienced.

One result of this work is to indicate that no patch smaller than about 10 sq. yd. should be placed. A patch of less area is too small to sustain the weight that comes on it. Also a patch of 10 sq. yd can be reinforced, while it is difficult to reinforce a smaller patch.

On this job it was found that a gang of about 20 men was about sufficient to keep the work moving continuously. With this size gang, an average of about 40 sq. yd. of pavement per day could be removed and replaced, the patches varying from 1 to 10 sq. yd. and extending along the road for a distance of 1500′ or ½ mile.

This gang would be distributed usually as follows:

- 2 men running drills
- 2 drill-running laborers as helpers.
- 4 men cleaning out broken concrete and excavating to required depth.
- 1 man setting forms.
- 2 men finishing concrete.
- 3 men shoveling sand and stone.
- 4 men wheeling concrete.
- 1 man operating mixer.
- 1 man watering concrete.
- 1 truck driver.
- 1 superintendent.
- 1 truck supplying sand and stone.
- 1 night watchman.

The cost of the work was as follows:

Removing old concrete and preparing opening for new patch	$1.19 per square yard
Replacing with new concrete, labor, mixing, placing, finishing, curing	$2.19 per square yard
Material for replacing, including delivery on the work	$2.73 per square yard
Total cost per square yard	$6.11

Block Pavements.—Block pavements of brick, stone, asphalt, etc. properly constructed should not require repairing for a considerable term of years. Cracks which develop should be grouted with hot paving pitch or asphalt binder. Areas which settle, thereby breaking the bond of the grouted joints, resulting in crushing or cobbling the blocks, should be taken up, the sand cushion reformed, all sound blocks cleaned and relaid and turned over where necessary, any broken blocks replaced by new whole ones, and joints then grouted with Portland cement grout preferably, if the original pavement was so constructed, otherwise the joints may be poured with hot paving pitch. It should be noted that repairs with fresh cement grout require protection by barricades for about a week, so that such repairs should be confined to one side of the pavement in long stretches, leaving the other side available for traffic; where the repairs are limited in extent and barricades are especially undesirable, the patch may be covered with 2″ of earth and further protected by planking during the time required for the grout to set. Where joints are poured with paving pitch, traffic need be diverted only during the time of actually making the repair; this is a decided advantage.

Rigid-pavement Shoulders.—Observation demonstrates that horse traffic on steep grades leaves the pavement and seeks the earth shoulder, so that, so far as practicable, these shoulders should be improved by widening, and by graveling or covering with broken stone to avoid excessive rutting; also that on sharp curves the tendency of motor vehicles is to cut close to the inner edge, making it well for this reason to stone or gravel the shoulders at these points.

Along the edges of the rigid types of pavement, block and concrete especially, traffic usually develops a deep rut which if neglected becomes dangerous to rapidly moving traffic; this rut should be kept filled with gravel, broken stone or cold patch. Excess material when removed from the shoulders should be so disposed of as to widen embankments and flatten slopes.

Snow Removal.—On main roads between large cities, snow removal in winter has become part of the regular program. In many districts automobile trucking relieves rail congestion and is needed even more in winter than in summer. The maintenance departments are in a position to handle this work with their organized forces and equipment which are idle at this time of year, and the necessary expense is certainly worth while to make the main road passable for trucks the year round. The only way successfully to handle snow is to start shoveling as soon as a storm starts and keep working. If bad drifts are permitted to form, it is very expensive to clear the road. Snow fences help to prevent drifting. In Monroe County, New York, the cost per mile per winter to keep the main roads passable for motors ranges from $50 to $100. The usual equipment consists of trucks or caterpillar tractors equipped with special plows.[1] Hand shoveling is at times necessary where bad drifts form. A permanent fund of $75 per mile per year is proposed with any unexpended balance for easy winters carried over for use in exceptionally hard winters. Four hundred miles are to be kept open in this county and the popular approval of expenditure for this work assures a continuation of this phase of highway maintenance.

MAINTENANCE OF EARTH, SAND-CLAY, AND GRAVEL ROADS

The maintenance of these roads consists in keeping the grass and weeds cut, the ditches clean, culverts clear, overhanging trees trimmed, and the surface of the traveled way scraped and dragged. One shaping with a blade road machine in the spring generally is all the heavy work required, the rest of the work being done with road drags, hones, planers, etc., at frequent intervals during the balance of the year. On sand-clay and gravel roads, surfacing material is added to fill holes and ruts or better the wearing surface.

There are two general systems: the contract system, which lets short strips of road not over 4 miles in length to farmers along the

[1] Each should be able to care for 15 to 25 miles of road with occasional extra help.

road, and the patrol system, which is taken care of by a steady patrol gang which handles from 10 to 20 miles. The contract system is explained in the quotation from the 1917 "Year Book" of the American Highway Association (p. 453). The patrol system is referred to throughout the chapter in various quotations.

Earth Roads.—Road machine blade scrapers are familiar to all readers. The road hones, planers, etc., are not so well known and their construction is shown in Figs. 179 and 180. Steel drags can now be obtained. Their use in earth- or gravel-road maintenance is explained in the following quotation from the "United States Forest Road Manual."

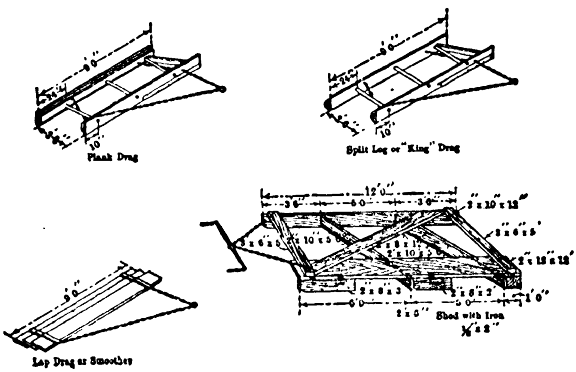


FIG. 179.—Road drags and planers.

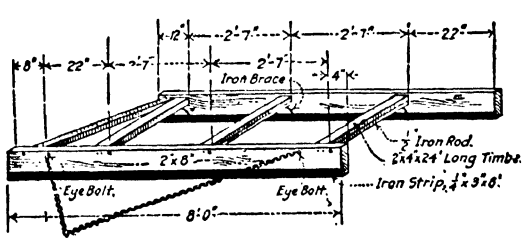


FIG. 180.—Road drag.

EARTH-ROAD MAINTENANCE (U. S. FOREST ROAD MANUAL)

"Maintenance is the most important item of work to be considered in road management. The smaller allowances for systematic maintenance, as they are being included in the annual road budget alongside the unlimited number of those for periodical repairs, tend to give it a place of least consideration, and again its consideration and planning are evaded as much as possible for these are the never-ending consideration of continuous annual work and expense. It is always admitted that the degree of efficient use we derive from anything constructed for practical utilization depends on the amount of effective maintenance it receives. Therefore, roads which are most widely used and exposed in all their parts to the worst of elements relatively should receive the highest degree of such attention, and, moreover, the higher the type of construction and the more it costs, the more marked attention will it require.

"However, it is most gratifying to find that the old ideas of taxpayer and user are rapidly disappearing, making way for the installation of practical system for the efficient retention of the better roads—as they are now being constructed. We are gradually beginning to learn that the stability and usefulness of a road are not forever established—even when the best of supervision and authorities declare and approve or have made the construction strictly up to the standard and with ample drainage provided—but that each mile of construction should be followed immediately with a mile of maintenance. Besides eliminating the difficulties and discomforts of travel—which seem only a benefit to the traveler, but which are in reality an economical benefit to everyone, directly or indirectly—maintenance will do away with all the worries to the management and effectively prevent so much of this misapplied criticism to construction features. Finally the good results of maintenance encourage more road building, whereas its lack discourages it.

"There is no type of road that can be considered permanent, and an earth road or one bedded in the natural material—which is wholly as important as the higher-grade roads, or even more so—is the cheapest to maintain in its original condition. The complete maintenance of an earth road means simply the retention of the drainage facilities that were provided in a completed and properly constructed piece of road work. Furthermore, the experience and the attention given to the road in constant maintenance will show where ample drainage was not sufficiently provided; and again showing its importance, constant maintenance secures this necessary drainage with the least costs and at the proper time—before serious damage is done and heavy repair costs result. The time to begin the maintenance is immediately after the road is constructed, and its degree of efficiency will depend on what, in the way of money or assistance, is constantly provided or made available to meet sudden contingencies. The work must be done at the right time and in the right way to get the best results

"Ample drainage begins with taking the water off the road and continues with taking it along the road and away from the road. Constant maintenance by dragging secures this primary step in the drainage system, and also a hard and smooth surface for travel. The dragging preserves the crown, which is kept in the traveled way for no other purpose than to shed water. It then follows that this water will be taken away from the road through the further efforts of constant maintenance in keeping the ditches and culverts open.

"To maintain a certain road or set of roads properly and economically, an organization for doing the work should be effected. On a country or mountain road a patrol consisting of two teams and two men for one part and one team and one man for the other part of the season should be able to care for 15 to 20 miles. It will be found though that a newly constructed road will require heavier maintenance for the first year or two, thus reducing the number of miles for this patrol. One or more such outfits could be applied to a longer road or a larger system and kept under the same supervision. These patrols keep the ditches clean and the culverts open, haul surfacing materials, *i.e.*, clay onto sandy portions and sand or gravel onto clay, keep the right of way open to sun and wind, and are on hand to drag the road after each rain. Two teams are provided only in cases where there is no extra help available along the road to assist in the dragging, otherwise one team would be sufficient. However, if the teams are government owned, two teams should be used, as the added costs for the extra team are small and will in most cases prove cheaper than hiring. The two teams can be used on one drag or two, depending on the ruling grades in the road.

"In early spring when the winter snows are going off, the supervisor and such extra assistance as is necessary should be made available early to see that the snow water is being cared for, that is, running down the ditches and into the culverts, and not down the wheel tracks and over the banks of the road. Later he should have a small gang of men making the necessary repairs that might occur while the frost is coming out of the ground, and from wash and water breaks. A light grader should be at hand, especially on side-hill roads, to clean the ditches of material broken off or rolled down the banks and to restore badly depleted crowns, after which the drag can be used for the remainder of the season to preserve this perfected condition.

"A good foreman for this should be a man who, as well as being able to take a hand in the work, should be able to plan the work and keep in touch with the maintenance needs and move his men economically to the first necessary piece of repairs.

"Dragging is the cheapest and most effective method of maintaining roads constructed of earth, topsoil, sand clay, or gravel. The drag is a very simple and inexpensive implement and when used properly gives surprising results.

"Properly used and at the right time, the road drag performs four distinct offices: (1) By moving at an angle to the traveled way it tends to produce or preserve a crowned cross-section; (2) if used when the material of the surface is not compact and hard, it tends to reduce ruts and other irregularities in the road by moving material from points which are relatively high to those which are relatively low; (3) when used after a rain, it accelerates the drying out of the road by spreading out puddles of water and thus increasing the surfaces exposed to evaporation; (4) if the surface material is in a slightly plastic state, dragging smears over and partially seals the so-called pores which naturally occur in earthy material, and thus makes the road surface more or less impervious to water.

"If used improperly or at a wrong time, the drag may do actual injury to a road. Dragging a very dry road, for example, serves to increase the quantity of dust and may do additional damage by destroying the seal produced during previous draggings. If, on the other hand, the road is very wet and muddy, the irregularities in the surface are likely to be increased rather than diminished. The common defect in road dragging is to regard the road drag as a road-building tool, and to expect one or two trips to put the road in shape for the season."

Notes on Maintenance

"1. In filling bad ruts and mud holes, it is best to use the same material that the roadbed is composed of, otherwise an uneven surface will result. Oftentimes, of course, the roadbed of clay can be improved by scattering sand or gravel over it more or less evenly, or, if the roadbed is of sand, by the same use of clay, but the ruts should not be filled with these applications. Filling with rock will effectively close a mudhole, but the next season will find two more mud holes, one on either side of this hard place formed by filling the first.

"2. After light snowfalls on side-hill roads, the inside ditches should be cleared of the snow immediately in order that the water from the melting snow will run down the ditches instead of the wheel tracks. This is especially necessary where steep grades occur to prevent heavy wash and loss of crown in the traveled way, and water breaking over the outside bank. As the snows are usually light, this can be done by drawing the drag down the ditch with a large skew angle or, better, with a small ditch cleaner, the *A*-drag or go-devil.

"3. In a grazing country very often it occurs that salting grounds have been used near or along the roads. These should be removed, for cattle climbing up and down the banks and walking along the ditches can cause considerable unnecessary damage to the road. During the season of cattle or sheep drives, the men on maintenance should see to it that the herds or bands, if they have to use the road, use the traveled way and not the banks, and do as little damage as possible. If serious damage is done, they can make immediate reports, as owners are obligated to repair such damages on public roads.

"4. Outer bank slopes of earth that are continually eroding should be protected by sowing to grass, or any other plant that will mat and not be objectionable to occupants of lands along the road.

"5. Keep the ends of the culverts free from drifting weeds and débris and clean the catch basins of silt and other deposits.

"6. Remember that the chief repairs should be looked after in the spring when the soil, moist and easily worked, will compact readily under the drag and traffic. There is little use in attempting to do much to the roadbed proper in July and August, for the soil is so dry that it is difficult to shape it properly and most of that moved will blow away in the first wind."

Notes on Dragging

"1. Use the drag often and if the very best results do not come at first trial, do not quit. First-class results can be attained.

"2. Dragging is always done after rains, melting snows, or thaws, just after the ground has lost its stickiness, when the material will slide easily along the face of the drag and pack well; but not when it becomes dry in any one place. Different road surfaces and varying conditions will demand different times of application, the knowledge of which will come through faithful and persistent use and observation.

"3. It requires a careful and skilful operator to get good and quick results, one who knows or can learn how to hitch to it, and where and how to ride it. Hitch so that the drag will travel at an angle of 45° with the center line of the road, and do not try to cut too much material at one operation. The amount moved depends wholly upon the length of hitch and position of driver. A long hitch will move more earth than a short one. When a hard spot must be cut, the driver throws all his weight on the front blade; when a low place must be filled he moves back. These operations on patented steel drags are facilitated by changing the angle of the blades from a vertical. Step quickly to the opposite end of the drag from which you wish to deposit material into low spots.

"4. Drive the team at a walk and ride the entire distance. The drag should begin at the ditch line and proceed toward the center or crown. If the crown becomes too great, reverse the skew angle of the drag. Do not try to drag too wide a section at one operation.

"5. Do not try to drag too long a section. So much depends on the time the drag is used, that there is danger of dragging the road too wet at one end and too dry at the other. Learn to select those sections which dry before the others and drag them first.

"6. Drag the road during or directly after one of the light snowfalls, just before it freezes up for the first time, as it will be in better condition to go through the winter and better able to shed water during the spring thaw.

"7. Very little improvement will be noticed after the first trial, and many trips will have to be made the first year after construction. The second year less dragging will be required and the road ought to improve continually

The following quotation from the 1917 "Good Roads Year Book" shows the Kentucky methods and approximate cost of maintenance.

"Maintenance by dragging is most successful when well organized. The results obtained by good management in Hopkins County, Kentucky, are frequently cited as indications of this, and for this reason the following account of the work there is quoted from a report by the Kentucky Department of Highways.

"In 1912 a county engineer was appointed. The county roads were measured under his supervision and 2-mile sections designated, and in January, 1913, drags were started on about 100 miles of the county roads. This original contract was only for dragging the roads, which work was to be done four times between Jan. 1 and Apr. 1, at a cost of $10 to $12 per mile. As the sections dragged were not continuous, the citizens at once appreciated the difference between the maintained road and that which was not maintained. Consequently, the next contract, which called for dragging and also for cleaning the ditches for 6 months, until November, 1913, resulted in contracts for 150 miles of road and at a reduced cost. In November, 1913, a contract substantially like that now in use was adopted and the time of the contract was for 1 year, or until November, 1914. Over 200 miles were maintained this year at an average cost of $28 per year per mile. For the year from November, 1914, to November, 1915, the benefit of the maintained roads was so well understood by the citizens that 560 miles were under contract at an average cost of $24.35 per mile per year.

"In November, 1915, a 2-year contract was entered into, which the county may revoke for a non-performance of the obligation at the end of the first year. About 520 miles are now under contract, at prices ranging from $12 to $40 per mile per year, the average being $22.10. It is expected this mileage will soon be increased. Originally, a contractor was allowed to have charge of 8 miles, but now he is not allowed to contract for more than 4 miles of road. Under the 1915 contracts, the contractor must trim the branches which overhang and interfere with travel on the roadway; keep the ditches clean, free from obstructions, and at all times capable of carrying the water. He shall, by June 1, each year, grade the roads with dump scraper, grader, drag, and ditcher, or in any way he may see fit, so that the center of the roadway shall be crowned so that the water will flow from the center of the road to the side ditches, and at no place will the water stand on the road or run down the road. The road shall be dragged from ditch to ditch at each dragging, when the road is wet, but not sticky.

"A record of the number of draggings is kept by the county engineer on cards which, before mailing to the contractor, are countersigned by the rural route carrier or a reliable citizen. The contractor also hauls material and constructs all culverts and bridges of 10′ span or under, and keeps the approaches to, and the floors and abutments of, all bridges and culverts on his road in good traveling condition. An analysis of these contracts shows that where the contract has been faithfully executed there is a decrease each year in the cost per mile, mainly because the farmer contractor has learned from experience that continuous maintenance makes a lower cost of time and labor each succeeding year."

Cost.—The cost of earth-road maintenance ranges from $20 to $200 per mile per year. A fair average is approximately $50 to $100 under light volume of traffic per mile per year for ordinary farming country and $100 to $200 per mile per year for mountain roads.

Sand-clay Roads.—The methods and character of work are the same for the sand-clay maintenance as for ordinary earth roads. The cost is generally less. The following quotation from the Alabama State Highway Report indicates the usual procedure.

Sand-clay Roads

"No cheap road can be maintained as easily and at as small an annual cost as a well-constructed sand-clay road. It responds readily to a road machine and the surfacing material is usually very convenient. Like all others, though, it is neglected until extensive and expensive repairs become necessary. If a sand-clay road which has been intelligently constructed is kept dragged at reasonably frequent intervals, say three times a month during December, January, February, March, and April, and during rainy periods in the other months, it will give excellent service and serve all practical purposes. If too much sand is in the surfacing material, the road will tend to ravel or disintegrate and it becomes necessary to add a small amount of clay to the sandy section. A thorough harrowing should then be given the surface, after which the road should be thoroughly machined or dragged until the proper cross-section is obtained. Likewise, too much clay may develop in wet weather and the addition of sand becomes necessary. Sand can be incorporated in like manner as the clay. In very wet weather, traffic will incorporate the sand fairly well and it frequently becomes necessary to add sand to prevent slipping, when artificial mixing would be difficult."

Gravel Roads.—Gravel roads require patrol maintenance for good results. The road should be shaped with a road machine blade grader in the spring while soft and plastic and kept in shape by dragging. Gravel must be added continuously to fill holes and ruts. Shoulder, ditch, and culvert routine cleaning is the same as for any maintenance.

The following quotation is from "Instructions to Patrolmen" in New Hampshire, which is famous for its gravel roads.

"Each patrolman must supply a horse and dump cart, shovel, pick, hoe, rake, stone hook, ax, iron bar, iron chain, and tamp. Special tools are furnished by the State Highway Department.

"One dragging in the spring is worth two in the summer. It is better to drag a mile of road several times and get it in good condition than to drag 2 or 3 miles and not finish any part of it. Don't drag a soft section when it is so wet that the first vehicle to pass will rut it all up. First fill the holes and ruts with new material and then drag as the surface dries out. Every patrolman should have material dumped in small piles along the side of his section so that on a rainy day he can at once fill all holes and ruts in which the water is collecting.

"When the weather is unsuitable for dragging, as during a dry spell, all patrolmen should cart on all the new material possible in order to fill all ruts and holes and resurface worn sections. Carting is very essential during dry periods and should never be neglected. Whenever a patrolman is in doubt as to what to do next, the general rule is to cart new material, for all roads are wearing out under travel and it is necessary that the surface be continually renewed to take the place of the old material that is thrown out as mud or blown away as dust.

"Save all the sods, leaves, rubbish, stones, and refuse that you clean off your road, and dump this waste material in places where the bank is steep so that by flattening the side slope there will be no need of a guard rail; or dump the material back of a present guard rail so that later this guard rail can be removed."

The necessity for patrol maintenance is shown by the following extract from the Iowa Specifications.

MAINTENANCE OF GRAVEL ROADS

"County engineers' and supervisors' attention is called to the fact that both Class *A* and Class *B* gravel roads require constant and systematic maintenance at all times. Special attention should be given such roads for the first year following their construction. During this period the gravel is sure to become rutted, wavy, and scattered if it is not maintained in the most careful manner.

"Hauling gravel and dumping it on the road does not produce a gravel road. The most important part of the construction work lies in the attention which the road receives while the gravel is being compacted. A road newly surfaced with gravel is nothing but a possibility. The success or failure of such a possibility depends very largely on the attention which it receives during its first year. The frequent use of a planer or blade grader will prevent the formation of ruts and waves. This work should be done while the gravel is wet, as better results will be secured.

"The scattered gravel should be brought back on the surfacing and the earth shoulders built up to hold this material in place. Additional gravel should be added to replace that worn away and to fill and depressions due to settlement.

"The Commission strongly urges that the patrol system of maintenance be adopted for all gravel roads. The patrolman should spend all his time on the road. It is only by such a system that definite responsibility can be fixed. Patrol maintenance should extend not only over the first year after the gravel surface is placed, but also throughout the succeeding years. It should extend to the side ditches, earth shoulders, culverts, and all other parts of the road as well as to the gravel surfacing.

"While the patrol system of maintenance is urged for all gravel roads, it is absolutely necessary for Class *B* gravel roads. These specifications have been prepared with that idea in mind.

"The Commission will approve the construction of Class *B* gravel roads on the county system only on condition that an adequate patrol maintenance will be established promptly after such road is placed in service," see page 353 for sections of Iowa Gravel Roads.

IOWA HIGHWAY COMMISSION.

GRAVEL-ROAD MAINTENANCE WITH "SPRING SCRAPERS"

Method employed in Kent County, Michigan, described in paper presented Feb. 14, 1923, at ninth annual Michigan Conference on Highway Engineering.

BY OTTO S. HESS

Road Engineer, Kent County

"The system of maintenance which will be outlined in this paper has been developed in Kent County during the past 2 years, and the ensuing remarks will apply more particularly to Kent County rather than to the state as a whole.

"While this system will undoubtedly operate to good advantage in a great many counties, it is not advocated by the writer as a cure-all for the many difficulties which are encountered in the maintenance of gravel roads.

"**Highways of Kent County.**—At the present time, Kent County has 252 miles of improved gravel roads on the county road system. Of this mileage, 90 miles lie on the state trunk lines, and the remaining 162 miles are county roads. In the early maintenance season of 1920, all of this mileage was taken care of by teams hauling light graders or floats. About this time, the county tried out a maintenance machine which was new at that time but quite well known throughout the state and even outside the state at the present time. This machine is the spring scraper which is used as a truck attachment and operates as a blade scraper underneath the middle of any truck.

"The first 'spring scraper' was so successful that the team patrols were gradually replaced by the so-called truck patrols, until at the beginning of the maintenance season of 1922 all of the teams had been replaced and the entire mileage of 252 miles of gravel roads was being kept smooth by nine trucks with scraper attachments, making an average of 28 miles per truck.

"**Work of Truck Patrols.**—Under the present system, Kent County is divided into four maintenance districts. The trucks are operated from a central point in each of these districts. A very thin layer of finely screened gravel is kept on the surface of the road at all times. By scraping this surface often enough it has been found possible to maintain a surface which is smoother to ride on than most pavements. The number of times per week or per day which is necessary to scrape the surface in order to keep it in this condition is dependent on the weather and the volume of traffic, but more particularly on the volume of traffic. The amount of scraping, and, consequently, the cost of maintenance have been found to be very nearly in direct proportion to the volume of traffic using the road.

"The type of trucks which has been found to be the most economical in Kent County are those in the 2- to 3-ton class. These trucks carry a 10′ blade, which is placed under the truck in a diagonal position, and travel at a speed of from 8 to 12 miles per hour. The blades are 10″ wide to start with, but are worn down with about 2 weeks steady use to a width of 3 or 4″.

"Each truck on scraping work is able to travel from 60 to 90 miles per day. Each truck scrapes a path approximately 9′ wide. With reference to the quality of work done, it can truthfully be said that the old team patrol system was never able to keep the roads so smooth as they have been maintained by trucks. In fact, the gravel roads of Kent County have never been in so good condition as they were in 1922, when they were maintained entirely by trucks.

"**Saving Effected by Substituting Trucks for Teams.**—Some idea of the saving which can be effected by substituting trucks for teams can be obtained by comparing the daily costs of each. In actual practice it was found that one truck would, on the average, replace six team patrols. In 1922, teams with drivers cost $6 per day, making $36 per day for six team patrols. One truck could be hired, with driver, at $20 per day. This shows a direct daily saving on the pay-roll of $16 per day per truck. Assume that the number of days worked during the season was 200 and we have: 200 days × $16 × 9 trucks = $28,000, which is the direct pay-roll saving during 1922. As a matter of fact and record, the actual saving was about $30,000 since Kent County operated its own trucks at about $15 per day instead of hiring them at $20. As a further saving which is effected by the scrapers, it should be remembered that it is necessary to buy and maintain only one scraper as compared with six outfits of teams. Another advantage, which is by no means unimportant, is the fact that the trucks can haul gravel or other materials and scrape the roads at the same time. This is a feature which saves many thousands of dollars in a year's time, and is not obtained by any other type of road maintenance equipment in use today.

"**Cost of Maintenance.**—In order that one may obtain some idea of what this maintenance work costs, the following table has been prepared, showing the cost per mile, volume of traffic, and cost per vehicle mile for all of the gravel roads on the state trunk-line system in Kent County:

MAINTENANCE COST OF GRAVEL ROAD ON STATE TRUNK-LINE SYSTEM IN KENT COUNTY

Section	Number	Mileage	Total cost	Cost per mile	Average daily traffic	Number vehicle miles	Cost per vehicle mile
13	1	7.50	$ 9,139.50	$1,218.60	1,791	2,686,500	$0.00340
13	3	7.00	10,634.76	1,519.25	1,810	2,534,000	0.00419
13	4	5.00	8,235.54	1,647.11	2,025	2,025,000	0.00460
13	5	14.50	12,077.29	832.11	838	2,430,200	0.00496
37	8	2.00	1,414.41	707.20	594	237,600	0.00595
37	10	5.50	3,819.89	694.53	594	653,400	0.00584
16	14	14.00	17,358.33	1,239.88	1,305	3,654,000	0.00475
39	16	11.50	5,836.71	507.54	594	1,366,200	0.00427
39	18	1.50	446.22	397.48	680	204,000	0.00218
44	20	13.50	12,380.47	910.77	1,079	2,913,300	0.00415
66	15	2.50	958.55	383.42	.....	No traffic count	
54	23	14.50	9,610.89	662.82	.....	No traffic count	
Totals, averages.....		99.00	$91,912.64	$ 928.31	.....	18,704,200	$0.00435

"In explanation of the table, the writer would advise that the total number of vehicle miles for any section was obtained by multiplying the average daily traffic (as determined by a traffic census taken in July and August) by 200, and then by the number of miles in the section, assuming that twice the average daily traffic in the summer would represent the total traffic for the year. This shows that the average cost per vehicle mile on the state trunk-line gravel roads was a little less than a half cent."

Cost.—F. R. White, Road Engineer of the Iowa Highway Commission, supplies the following information in regard to the construction and maintenance cost of about 400 miles of Class *B* gravel roads (see Fig. 115 p. 353). These roads are constructed at a cost slightly above $1000 per mile (1913). The cost of maintenance depends very largely on the volume of traffic and the location of gravel. Where there is an average of 200 to 300 vehicles per day, however, and the gravel can be obtained within 3 miles of the road the yearly cost of maintenance is about $150 per mile (1916).

In New York State where the roads are oiled to care for a somewhat larger volume of traffic, 200 miles of high-class gravel roads cost approximately $700 per mile per year to maintain.

A fair average maintenance cost per mile per year for double-track gravel roads is probably from $200 to $300 under fairly heavy traffic (300 to 400 vehicles daily).

CHAPTER VIII

RECONSTRUCTION

Resurfacing or reconstruction is resorted to when normal maintenance methods with reasonable yearly expenditures fail to keep the highway in proper condition for traffic. Economic limits of yearly maintenance costs and maximum permissible degrees of roughness were discussed in Chap. 7 (pp. 547 to 552).

Reconstruction includes the restoration or improvement of pavement conditions in regard to strength, width, and perfection of surfaces, the retention of a safe and convenient grading section, ditch to ditch, and sometimes considers alignment changes to increase safety and reduce objectionable grades. Relocations in the matter of alignment and grade were discussed in Chap. II. Existing practice in pavement reconstruction generally fulfils the principles of improvement of surface and pavement width but often neglects the item of the retention of a safe grading cross-section ditch to ditch. That is, proper shoulder and ditch work is often slighted, which results in added danger from narrow soft shoulders and deeper ditches. Existing ditches are often deepened to get shoulder material instead of providing an item of borrow excavation for suitable gravelly soil for the necessary shoulder fills.

The cost of reconstruction programs is affected by type selection and by the promptness with which roads are treated when they show the preliminary evidence of some weakness. Delay, due to shortage of funds, is probably the most active cause in raising reconstruction costs (see p. 582).

Figure 181 shows typical reconstruction designs, state of California.

Economic Type Selection.—Reconstruction programs are overshadowing original construction in many of the states having state-aid programs over 15 years of age. The economic selection of pavement type for reconstruction is a proposition entirely different from the selection for original construction. Economic reconstruction must utilize to best advantage the existing pavement. To do this, it is generally undesirable to change the type of foundation, although it is often desirable to strengthen the foundation and better the surfacing, particularly to get rid of oiling maintenance on main roads; that is, if the existing road is of macadam type serving traffic of Classes II*A*, II, or III, it is rarely desirable to change to the rigid type of pavement, although it may be desirable to strengthen the existing macadam and surface with an asphaltic concrete or block pavement in order to eliminate the oiling nuisance. If an old macadam practically shot to pieces serves Class I traffic, it is evidently desirable to change type.

All original roads become in time merely foundations for easily repaired renewable surface courses which meet the demands of

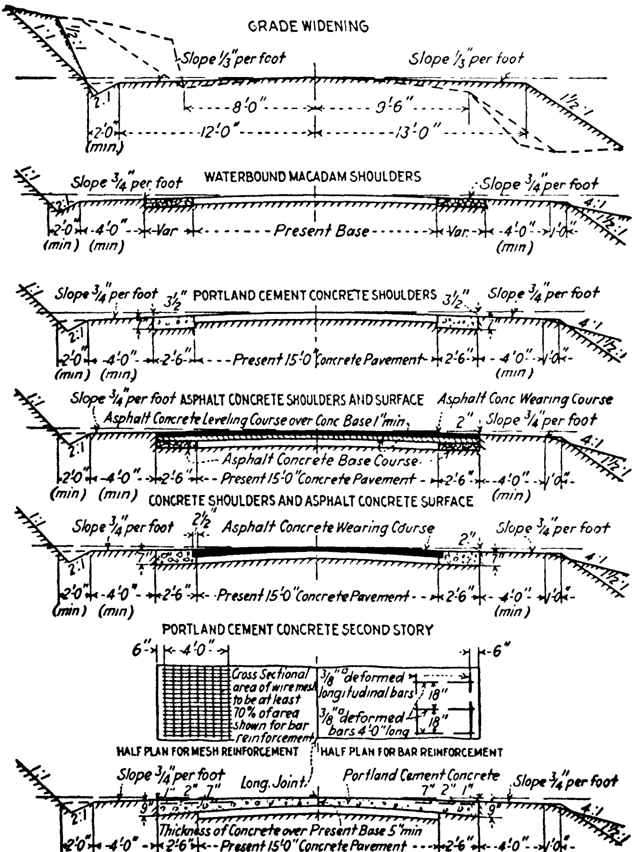
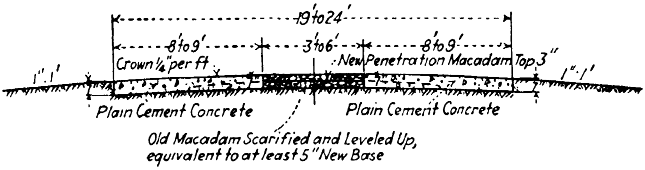
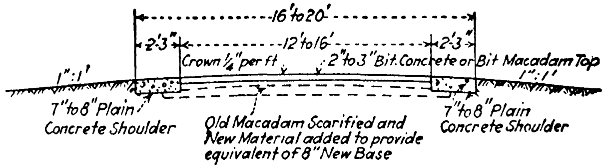


FIG. 181*A*.—Reconstruction sections, state of California 1926. Widening and thickening old state roads.

traffic. Adequate foundation strength can be achieved either by sufficient depth of macadam or sufficient depth and strength of cement concrete. After adequate foundation strength is attained by either method, satisfactory surfaces can be maintained at about the same cost per year. As discussed in Chap. VI, Design, the thickness of cement concrete foundations is about the same whether laid on old macadam or new grading; where an old macadam pavement exists and the reconstruction type is changed from a flexible foundation to cement concrete, most of the value of previous pavement work has to be thrown away which usually raises the cost of such work way beyond the method of retention of foundation type with necessary additional strengthening and improvement of surface.[1] To illustrate briefly from Division 4,



RECONSTRUCTION AND WIDENING OLD SURFACE



RECONSTRUCTION OF OLD MACADAM AND WIDENING BY ADDITION OF CONCRETE SHOULDERS

FIG. 181*B*.—Reconstruction sections. U. S. Bureau Public Roads 1926.

New York State, 1922 cost conditions: Old, firm, water-bound roads serving Class III traffic can be resurfaced with water-bound macadam for about 80 cts. per square yard, or with penetration bituminous macadam for about \$1 per square yard. If type is changed and the thinnest, cheapest rigid pavement possible is used, 6″ reinforced cement concrete, it costs about \$2.70 per square yard. A good many of the old water-bound roads are too weak

[1] The depth of concrete bases or pavement laid on top of an existing old macadam road is about 1″ less than where laid on ordinary earth subgrade (see Chap. VI). This reduces the cost of the concrete pavement about 50 cts. per square yard, whereas the old macadam, if utilized to full advantage as part of a flexible-type pavement, is worth about \$1.50 per square yard.

for modern traffic and require additional strengthening over and above a new 3″ top. This additional depth rarely requires more than a 3″ to 5″ middle reinforcing course, costing from 40 to 70 cts. per square yard. For such roads, retention of water-bound type with adequate strength costs about $1.20 to $1.50 per square yard and adequate strength with improved penetration-bituminous macadam top costs from $1.40 to $1.70 per square yard, while change of type costs at least $2.50 to $2.70 per square yard. Under this class of traffic, the difference in first cost far outweighs the minor difference in yearly surface maintenance of the two types (see Maintenance Costs, p. 520).

Under Class II and II*A* traffic, firm macadams can be resurfaced with penetration macadam for about $1.20 per square yard, and with asphaltic concrete or small cubes for from $1.50 to $1.80 per square yard. Change of foundation type costs about $2.70 to $3.40 per square yard. For the weaker macadams, the cost of proper reconstruction, using necessary additional reinforcing middle course, probably averages about $1.90 per square yard for penetration macadam and about $2.20 to $3 for low-maintenance-cost surfaces, such as asphaltic concretes and stone or brick cubes. The advantage still lies with retention of foundation type, considering the sum of reconstruction costs and yearly maintenance.

Under Class I traffic, the advantage falls on the side of change of type, and for these roads there is no hesitation in adopting the policy of rigid foundation type. In this territory, however, there are at present 860 miles of improved roads of all types serving all classes of traffic. Table 110 below indicates the relatively small mileage on which it is desirable to change foundation type, namely, about 5 to 15% of total mileage at present constructed; but this percentage may be radically changed by delay in necessary reconstruction (see p. 583).

TABLE 110

Class of traffic	Flexible type of foundation, miles	Rigid type of foundation, miles
Class I	42[a]	34
Class II*A*	106[b]	72
Class II	280	58[c]
Classes III and IV	220	48[c]
Totals each type	648	212
Grand total, 860 miles		

[a] Mileage on which it is desirable to change foundation type, 5% of total.
[b] Mileage in the doubtful class 12% of total. Total mileage on which there is some justification for changing type, 17% of total constructed.
[c] Suitability of original design doubtful.

Effect of Delay in Reconstruction on Final Cost.—The costly result of delay in resurfacing at the proper time is shown by the following official report.

Apr. 9, 1923.

"Mr. ——————,
Division Engineer,
Rochester, N. Y.

"Dear Sir:

"Find attached design report for the proposed Federal Aid Reconstruction of Monroe Avenue, Road 94, Sta. 129+27 to 219+80.4 (1.71 miles).

"Two estimates have been made. Design 1 calls for an 18′ width of cement-concrete pavement 8 by 6½ by 8″ thick. Mesh and bar reinforcement with special shoulders 3′ wide on both sides, constructed from the old macadam excavated from the center of the road. Design 1 is estimated to cost $78,000.

"Design 2 calls for an 18′ concrete pavement 7″ by 6½″ by 8″ thick with 2′ width of gutter and a curb 7″ high along the left side of the pavement, Stas. 129 to 184 where the Rochester & Eastern Electric Ry. tracks are located close to the pavement edge. A special shoulder 3′ wide constructed of excavated macadam is used on the right side. From Stas. 184 to 219 the design remains the same as for the first case. This design gives an effective pavement width of 20′, Stas. 129 to 184, and provides a raised curb protection next to the track. Design 2 is estimated to cost $88,000.

DETAIL REPORT

"Location and Length.—The proposed reconstruction extends from Clover Street to the end of the brick pavement near the Barge Canal (Stas. 129+27 to 219+80.4), a distance of 1.71 miles, of which 0.12 mile is in the town of Brighton and 1.59 miles in the town of Pittsford. This proposed reconstruction covers the portion of Monroe Avenue which is at present in poor condition. The balance of Road 94 between Rochester and Pittsford is in very fair shape for traffic.

"Previous Designs and Recommended Treatment.—The portion of Road 94 covered by this contract has needed attention since 1918. In 1920 a survey was made and two alternate designs submitted. The recommended design of 1920 provided for widening the old 14′ macadam to 16′ with a slag-macadam extension and a new surface course of 2″ depth of Topeka Mix on a 1½″ average depth of binder evener course. Slag-macadam shoulders were provided on both sides of the pavement (3′ wide), giving a total width of hard surface of 22′. This design was estimated to cost $50,000. An alternate design was prepared but not recommended, using 6″ uniform depth of cement concrete 16′ wide at an estimated cost of $60,000.

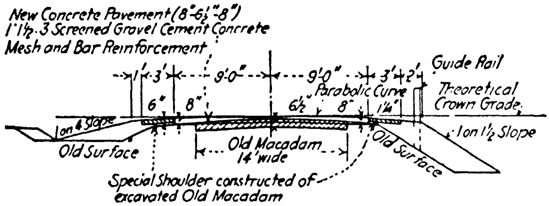
"At the time these estimates were made (1920) the old macadam was firm but rough, and the additional 3½″ should have provided the additional strength needed to prevent deterioration under the growing volume and weight of traffic. During the last 3 years, however, the value of the old macadam as a base course has been greatly reduced, as the lack of the additional 3½″ proposed has permitted churning under heavy loads which would not have occurred had the pavement been strengthened at the proper time. This loss of effectiveness due to breaking of the bond makes it advisable to eliminate consideration of the use of the old macadam as a base for an asphaltic-concrete top, and makes it desirable to construct some kind of a rigid pavement. Considering limitation of funds, cement concrete is probably a reasonable solution for this contract.

"The only way any practical value of the old macadam can be procured is to drop the new pavement into the old road and use the excavated macadam for shoulders close to the concrete. This has the double advantage of reducing fill and getting a firm solid shoulder at small cost. This is the general basis of the present design.

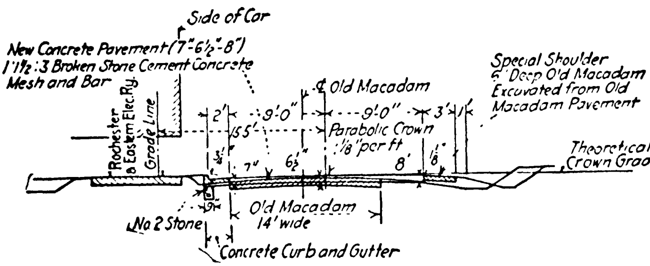
"It can readily be seen that the delay in reconstruction due to shortage of funds has greatly increased the cost of the necessary work.

"Recommended Section.—The chief problem in the design is the matter of drainage along the car tracks, Stas. 129 to 184. The best method of treatment is Design 2, with the curb and catch-basin construction. If the curb is not used, J. E. Kelley has made the excellent suggestion of using a one-way crown away from the track, which will help in the matter of surface-water seepage under the left edge, but will not entirely eliminate this trouble. On account of this seepage and the poor character of the soil on the outer edge of the old shoulder, an edge depth of 8″ for Design 1 is recommended. The center gets some benefit from the old macadam and less seepage softening, and a 6½″ edge depth is recommended. For Design 2, an 8″ edge depth is recommended where no gutter is used, and a 7″ edge depth where gutter is constructed. The center depth is the same as for Design 1. Excavated macadam is to be used for the new shoulder next to the concrete.

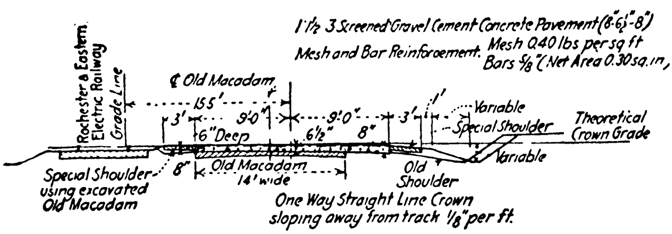
"**Culverts.**—The existing culverts on this road are the old-style vitrified-clay pipe and are, as a rule, in poor condition. New cast-iron pipe has been designed.



Alternate Design Sta. 129 to 184.



Recommended Design Sta. 184 to 219.



Recommended Design Sta. 129 to 184.

FIG. 182.—Recommended and alternate designs reconstruction road 94 (1923 design—estimated costs $88,000 and $78,000).

"There are two old masonry structures which should be pointed up, given new footing courses, and lengthened by the town of Pittsford.

"The Rochester & Eastern Electric Ry. should be required to provide the necessary outlet structures under their tracks to take care of the large flow at culvert locations, and to handle small ditch flow at intervals of not exceeding 300′.

"**Alignment and Grade.**—The alignment is excellent. The sharpest curve is 5°. No new right of way will be required.

"The grades are easy, 5 % maximum.

"The grade line as laid will provide enough excavated old macadam for the new proposed special stone shoulders. For the new earth shoulder widening both borrow and overhaul will be required.

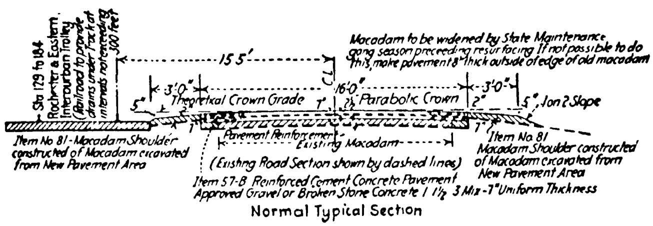
"**Borrow.**—Borrow can be obtained from the Barge Canal spoil banks within 2000′ of Sta. 219, or from the old tow path of the old Erie Canal west of Lock 32. The first source is the more feasible and has been used in computing overhaul.

"**Materials.**—Stone and sand must be imported, delivery (West Shore Ry., Pittsford) 1000′ from Sta. 220 of this contract.

"Water, Lake Ontario Water Company, hydrant at Sta. 136 and new proposed Long Meadow pipe line, Stas. 190 to 219.

"Cement, Pittsford delivery (West Shore Ry.).

(*Signed*)
Designer."



Recommended design reconstruction road 94 (1920 design estimated cost $50,000).

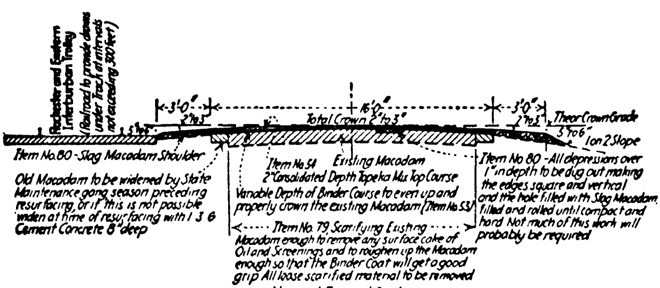


FIG. 183.—Alternate design in 1920. Estimated cost $60,000.

Note.—A comparison of figures 182 and 183 shows the effect of delay in reconstruction cost due to serious deterioration of old macadam.

Suitable Surfaces.—The suitability of surfaces has been thoroughly discussed in Chaps I and VI. The same conclusions hold, namely, that for Classes III and IV water-bound macadam oiled, or gravel well maintained by dragging and oiling, are satisfactory under all conditions. These surfaces are satisfactory on any steep grade occurring on improved road systems. For Class II traffic, water-bound, penetration bituminous macadam, asphaltic concretes, cement concrete, or any standard block or small cube is satisfactory. The limitations of use on account of grade have been discussed (see pp. 425 and 101). Standard-sized paving brick, asphalt block, etc. are not so rational as the smaller units on a flexible base and they also increase the difficulties of shoulder work.

For Class I, any standard type of high-grade surface is suitable under limitations of local preference and final cost discussed in Chap. VI. At the present time, the most easily used type for Class I resurfacing is some type of asphaltic concrete, but on steep grades this is not very satisfactory and it is hoped that the brick or stone manufacturers will produce a satisfactory smaller unit-sized block for these conditions. On main roads it is desirable to use a surface which is not maintained by surface oiling, as the oiling process is disagreeable for traffic for a short period each season. On secondary roads, short-time annoyance of surface oiling is far outweighed by the reduced total cost of the macadam type, that is, very few communities can afford perfection of comfort on secondary highways.

RECONSTRUCTION DESIGN

General Principles of Design.—Each road is a special problem requiring a complete knowledge of the strength of the existing pavement and careful alternate estimates of different methods of treatment. There are an innumerable number of solutions which serve the purpose, provided the general basic principles are not violated. A few typical examples are given to illustrate common cases. Basic principles of reconstruction design are listed as follows:

1. Make all necessary alignment changes on the score of safety or reasonable maximum grades (see official report, p. 594).
2. Adequate pavement width is essential and should be provided at this stage of programs.
3. Retain the same general type of foundation unless existing pavement is practically worthless.
4. Strength design for either the macadam or rigid type is determined in the manner discussed in Chap. VI. High-type surface should never be used on a weak base. This is often done.
5. Suitability of surface depends on traffic demands. On heavy-traffic roads, avoid a surface which requires oiling.
6. Retain same type of surface where existing pavements are to be widened only.
7. Avoid use of so-called dual types unless unusual conditions, such as central car tracks, prevail.
8. Safe and solid shoulders are essential.
9. Avoid deep, dangerous ditches. Use storm-sewer systems if possible to eliminate dangerous ditches.

Utilization of Old Macadam or Gravel.—An old macadam pavement must never be scarified unless the loosened material is entirely removed and used for shoulder material on the side of the new surfacing. Scarified macadam never can be put back into a good hard lock as the mechanical bond of the larger stone fragments is broken and they are completely separated by the screenings. Scarified macadam is nothing but a stone gravel which cannot have the stability of the interlock construction. Inequalities and shape of crown must be corrected by adding new material.

If an old macadam pavement is to be widened and used as a base for a high-type surface, the widening should be done by state maintenance forces a year before the resurfacing, in order to give it a chance to harden under traffic. If the widening and new high-grade surface must be constructed hurriedly, the most feasible solution is to use 1:3:6 concrete about 8″ deep for the foundation widening in place of macadam, although this method is not advised nor is it often necessary with reasonable forethought. If an old macadam is to be widened and recapped with macadam on Class II or Class III roads, all the work may be done the same season, although it is desirable to complete the widening well in advance of the new top and give it considerable traffic pounding to better the consolidation.

If an old macadam pavement must be strengthened before being recapped with a high-grade asphaltic concrete or block surface, the resurfacing additional thickness of macadam must be constructed a year ahead of the final resurfacing to give it a chance to harden under traffic. Asphaltic concretes or block surfaces must never be laid on new macadam which has not been pounded by traffic for at least 6 months.

All potholes must be dug out at least 3″ deep, the edges squared up, filled with 1¼ to 2½″ stone or slag, thoroughly rolled, filled with slag or stone screenings, and well compacted. Cold patch should not be used for work of this kind. Potholes must never be filled by varying the depth of the macadam of the overlying course.

The reinforcing middle course must not be laid until all potholes are completely repaired. This course must be fairly uniform in thickness; there should be no sudden variation of over 20% in finished depth.

The top course must be very uniform in thickness. Macadam top should have no sudden variation of more than 15%, and asphaltic-concrete surface mix no sudden variation of more than 10% in depth. These rules in regard to uniformity of depth are violated quite frequently and explain many unsatisfactory repair jobs.

For method of utilizing existing macadam where a new rigid-type pavement is to be used, see pages 583 and 585.

Case 1. Widening Existing Pavements.—This case is illustrated by Figs. 184 to 185, Road 5 near Rochester, N. Y., carrying 4000 to 5000 vehicles daily (10-hr. count in summer). This road is an asphaltic concrete (modified Topeka) on concrete base, Stas. 0 to 74, and an Amiesite top on macadam base, Stas. 74 to 130. The foundation soil is sand and gravel. It serves traffic well with the exception of width. Figure 184 shows the first design prepared, which violated all the basic principles of reconstruction design. It changed general type of foundation and used different types of surfacing. Figure 185 shows a new design which not only retains type but is cheaper in construction cost. Figure 185 is a satisfactory design for a case of this kind, except that the concrete base should be thicker than shown (for recommended depths, see Table 86, p. 426). The object of the undercut concrete base connection is to prevent separation of additional base from original base; it gives a good grip contact.

Case 2. Recapping a Worn-out Rigid-base Pavement Having a Removable Top.—This case is very simple; it merely involves the removal of the old, worn-out surface and its replacement by any standard type of surface desired. The old rigid base is retained and repaired if necessary. New edging or stone shoulders bound with bitumen are provided. This case does not require illustration by typical sections.

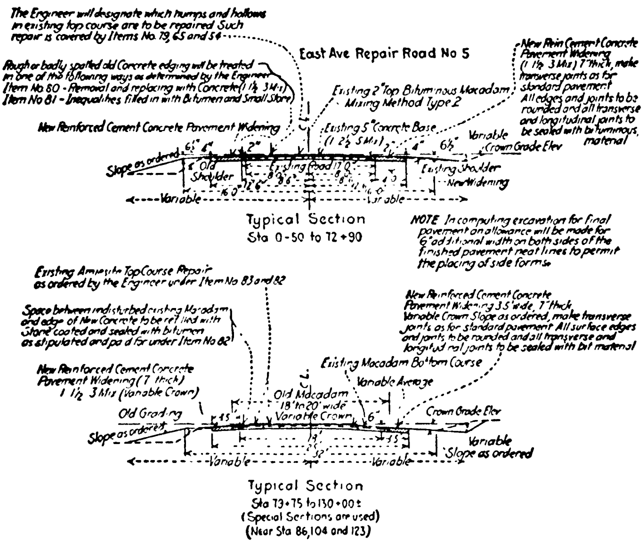


FIG. 184.—Design 1, Road 5, this design violates all principles of reconstruction.

Case 3. Recapping a Worn-out Monolithic Rigid Pavement of Ample Strength and Width.—Broken-down or weak areas of the old pavement are cut out and removed and the edges of the retained pavement squared up. The subgrade is undercut along the connecting edge similar to Fig. 186; new concrete base is laid and well tamped into the undercut. This concrete is protected from traffic and allowed to set 14 days before the new surface is placed. Two general types of surface can be used, asphaltic concretes, or any standard block or cube form of pavement.

If asphaltic concrete is used, the procedure is essentially as follows: Potholes in the old pavement over ½″ deep, but not too deep to weaken the base, are chiseled out to a square edge and filled with coarse-stone cold-patch material filled with coarse sand and thoroughly rolled. Small depressions, cracks, and crown irregularities are cleaned out and filled by varying the depth of regular binder course. For this purpose, an average depth of 1 to 1½″ compacted binder is generally sufficient. The final surfacing can be constructed of any desired mix of asphaltic concrete ranging in depth from 1½ to 2½″ as desired. For Class II or II*A* traffic, 1½″ modified Topeka makes a good surface; for Class I traffic,

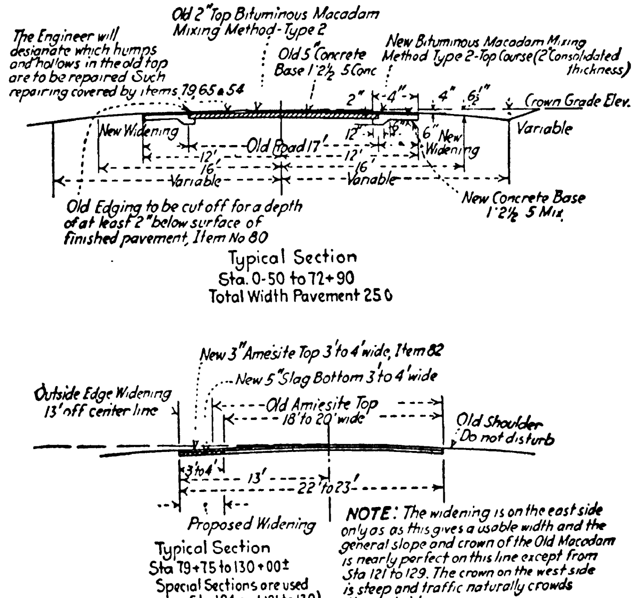


FIG. 185.—Design 2, Road 5, this design complies with basic design principles and is cheaper than design 1.

2″ of fine aggregate sheet asphalt makes a good surface. The thickness of the surface course must be uniform (see p. 587). Well-compacted bituminous macadam shoulders at least 4 to 6″ deep and 2′ wide should be constructed along the edge to prevent excessive edge wear of the high-class pavement. Safe width of grading shoulder and safe ditch section should be provided. This case is illustrated by Fig. 186.

If a standard block surface is used, the repair of the base for weak areas is the same as described. Pothole repair for depressions over ½″ deep consists of chiseling out to a square edge and filling with 1:2:4 or 1:½:3 concrete which is protected and allowed to set for 14 days. Cracks, minor irregularities of crown and small depressions are cleaned out and removed by varying the depth of the cement-sand cushion on which the new surfacing is to be laid. An allowance of 1" to 1½" average depth of 1:4 cement-sand cushion is generally ample to meet this requirement. On this cement-sand cushion, the block surface is constructed in the same manner as for original construction. The edge of the pavement

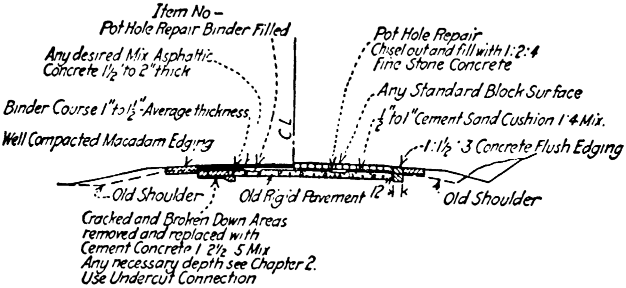


FIG. 186.—Typical design, Case 3.

should be protected by a simple concrete flush edging outside of which the stone shoulder, earth grading, and ditch section are the same as for the previous case. This case is illustrated by Fig. 186.

Case 4. Change of Type on Class I Traffic Road. a. Where Old Macadam Is of Ample Width.—Drop a new rigid pavement below surface of old, rough macadam in order to get a uniform supporting surface and in order to obtain cheaply the stone shoulder needed on each side of the new rigid pavement; that is, the old

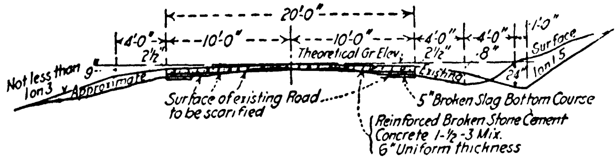


FIG. 187.—Case 4, example of poor design.

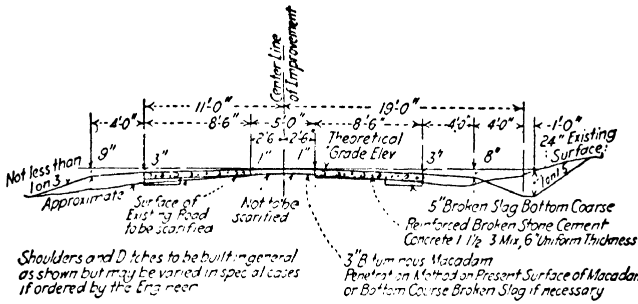
macadam top course is scarified and removed and used as a shoulder along the sides of the new pavement.

The new pavement should be of ample depth for macadam support conditions (see Chap. VI on Design, for the effect of macadam underlying support).

b. Where Old Macadam Is of Less Width Than New Pavement.—The same procedure applies in regard to scarifying, removing old top, and utilizing it for new shoulder. In designing depth, it is good policy to increase edge depth over middle depth where the advantage of macadam support is lost. It is poor design to attempt to widen the macadam and then use uniform depth of new pavement (see also Chap. VI for relation of edge to center depth).

Figure 187 illustrates poor practice. Figure 182 illustrates good practice.

Case 5. Three-strip Dual Type of Construction, Class I Traffic (Not Recommended).—The general idea of this type of reconstruc-



Case 5. Dual type (poor practice).

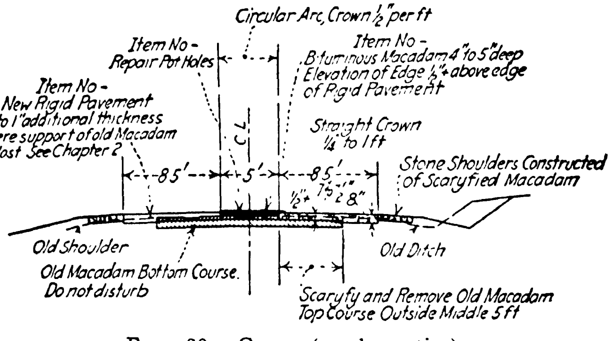


FIG. 188.—Case 5 (good practice).

tion is shown in Fig. 188. Mixed types are generally considered poor design, as they are expensive to maintain, although the dual type is quite popular with traffic, as it tends to separate the lines of travel.

The same principles of utilizing old macadam top course for shoulder material applies as in Case 4; the same principal of deepening the pavement along the edges where it loses macadam support applies as in Case 4. Case 5 is illustrated by Figs. 188 (poor practice) and 188*A* (good practice).

Case 6. Recapping Firm Rough Macadam.—Potholes in the surface over ½″ deep are picked out for the full depth of the top course, and the edges squared up. The hole is filled with regular top stone or slag (1¼″ to 2½″ size), thoroughly rolled to a tight lock, filled with coarse sand or screenings, and hard rolled. The surface course is then laid in the usual manner, using any type desired. Water-bound or penetration bituminous macadam 3″ consolidated depth can be used for Class III traffic. Penetration bituminous 3″ deep is satisfactory for Class II traffic. Some form of asphaltic concrete, preferably of the coarse aggregate type averaging 2½ to 3″ finished depth is satisfactory for Classes II or II*A*. Coarse-graded Amiesite is particularly adapted to such cases, as it adds materially to the structural strength of the old base. Under the heavier II*A* traffic, a 1″ binder course with 1½ to 2″ of modified Topeka is a good solution, provided steep grades are not encountered. The thickness of the top course must be uniform. On steep grades, the single-pour bituminous macadam or small cubical block surface is used. If a block or cube surface is used the minor surface irregularities of the old macadam remaining after the potholes have been repaired are eliminated by a varying depth of 1:4 cement-sand cushion; joints are filled with either bitumen or sand

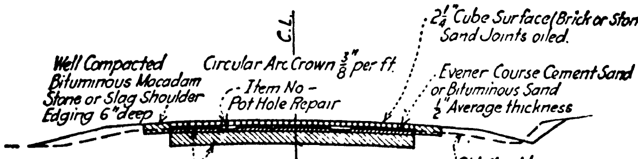


FIG. 189.—Recapping old firm macadam with cube surface.

with a light surface oiling. The new shoulders should be a good gravelly soil and the grading width of new shoulders and ditches must be safe.

Dont's.—In recapping an old, firm, rough macadam, do not scarify the old surface and add material (see p. 587). If the old surface is a smooth, slippery, flush-coat, bituminous macadam on which it is difficult to get a good grip for the surface coat, it can be either scarified and completely removed and used on the shoulder, or a 3½″ depth of new top can be used which has considerable internal stability of its own, or a bituminous paint binder can be applied overlaid with an open mix binder about 1″ deep and finished off with a Modified Topeka surface mix.

Don't attempt to widen an old macadam base the same year that it is recapped with a high-grade asphaltic-concrete or block surface for Class II*A* or Class I traffic. Widening for this class of work should be done the preceding year. Under Class II or Class III traffic with an ordinary macadam recapping, this restriction does not hold. If a quick widening is absolutely necessary for Class I or

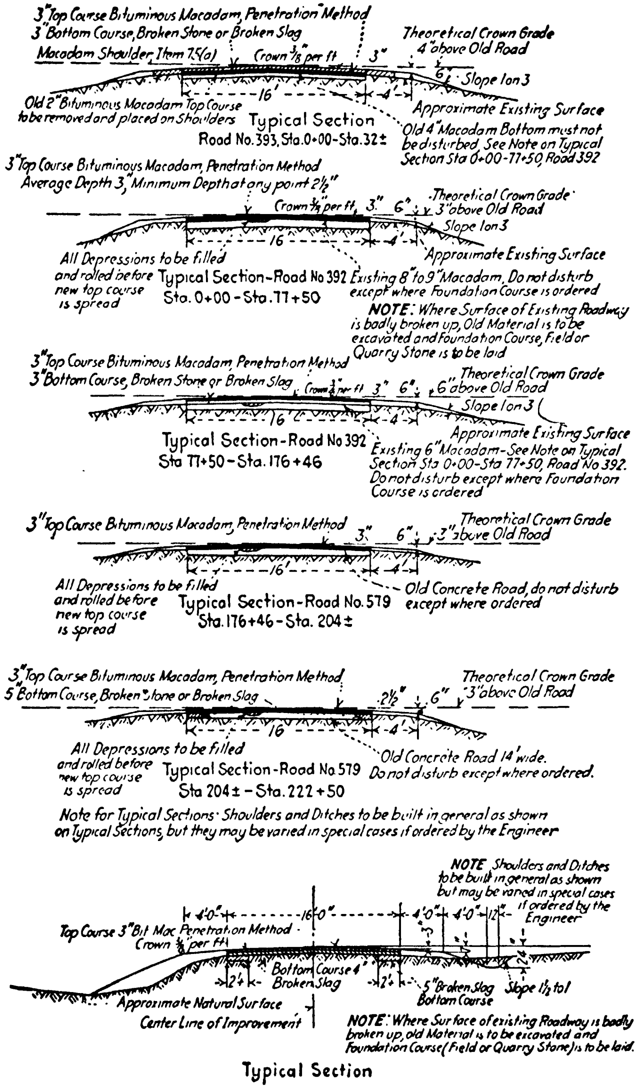


Fig. 190.—Case 7. Good typical example of widening, strengthening and recapping an old narrow weak macadam pavement. Division 4, New York State.

Class II*A* traffic, a depth of 8″ of 1:3:6 cement concrete for the additional foundation width is probably the best solution.

The final top course must be uniform in thickness (see p. 587).

Case 7. Reinforcing and Resurfacing an Old Weak Macadam, Traffic of Classes IIA, II, or III.—Actual conditions as to underlying soil and existing depth of macadam must be thoroughly investigated by test holes and records of spring blow-ups. The portions of the road which are entirely inadequate should be completely removed and rebuilt to the proper depth (see Chap. VI, p. 391). For this work, regular construction methods apply, utilizing any available type of subbase or foundation course Where a middle reinforcing course of 3 to 5″ depth in conjunction with the new surface course will produce an adequate depth, the old road is left untouched. Potholes are repaired as in Case 6. These must be repaired before the reinforcing course is laid. The reinforcing course is essentially the same as ordinary macadam bottom, and ordinary construction methods and materials requirements apply. The top-course construction is the same as for new construction where water-bound or penetration macadam is used. If a high-grade asphaltic concrete or block surface is contemplated under Class II*A* traffic, it must not be laid the same season as the reinforcing course. Traffic must be permitted on the reinforcing course for at least 1 year. Under these conditions, and reinforcing course should be bound with screenings instead of sand and raveling prevented by applications of calcium chloride (Case 7 illustrated by Figs. 189 and 190).

TYPICAL REPORT ILLUSTRATING ADVANTAGE OF RECONSTRUCTION RELOCATIONS

Rochester, N. Y.
May 16, 1923.

"Mr. ——————,
Division Engineer,
Rochester, N. Y.

"Dear Sir:

"Find attached detail report on the proposed relocation of Clover Street Road 294, Stas. 0 to 9 (canal crossing).

"We recommend a 16′ bituminous-macadam pavement on straight alignment with 5% maximum grade at an estimated cost of $9500. If cement-concrete pavement is used the cost is estimated at $11,800.

"This relocation is justified on the score of safety alone, as it eliminates two sharp curves, an 8% grade, and an old narrow, unsafe bridge. The reduction in motor operation cost on the new line probably warrants a construction expenditure of about $10,000, but this consideration need not be given much weight for this particular project, although it furnishes additional argument in favor of the work.

Sincerely yours,
Designer."

DETAIL REPORT

Clover Street Road 294 (Canal Crossing)

"**1. Location and Length.**—The proposed relocation extends from Monroe Avenue Road 94 to Sta. 9+00 of Clover Street Road 294, a distance of 900′.

"**2. Traffic Classification.**—Road 294 is on the borderline between Class II and Class III traffic. The 1920 census gives 550 vehicles in 12 hr. summer travel and the 1922 census 400 vehicles in 12 hr. For purposes of estimating the economic value of the proposed improvement an allowance of 800 vehicles daily will be made.

"3. **Grades and Alignment.**—The proposed relocation materially improves the grade and alignment. The existing alignment has two sharp curves, one of 75′ radius and one of 200′ radius. The existing grades are steep, and an 8% bridge approach grade is the maximum. The proposed alignment is straight with a 5% maximum rate of grade. The distance on

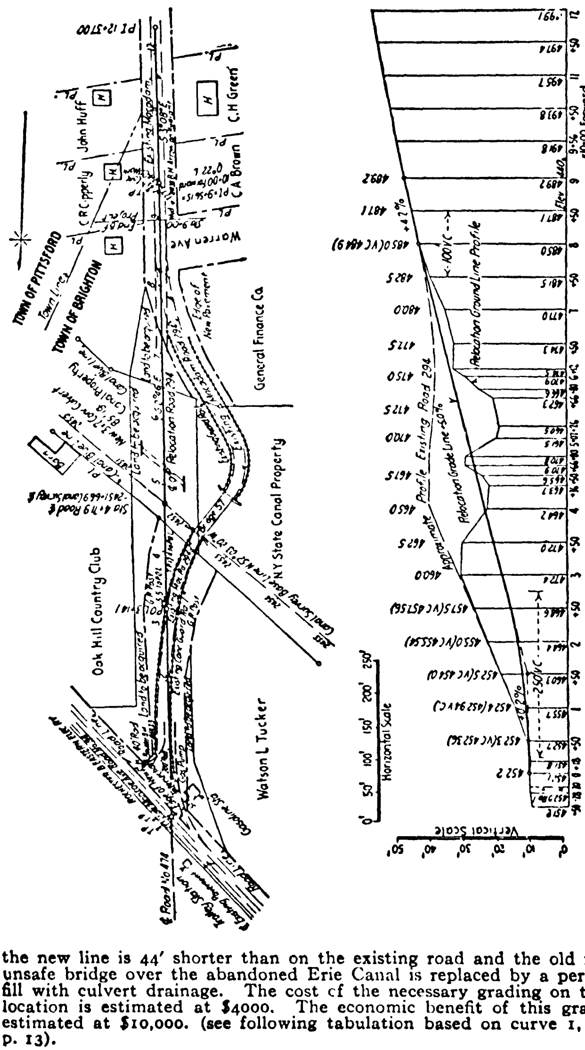


FIG. 191.—Plan and profile to accompany official report illustrating the value of the Clover Street relocation.

the new line is 44′ shorter than on the existing road and the old narrow, unsafe bridge over the abandoned Erie Canal is replaced by a permanent fill with culvert drainage. The cost of the necessary grading on the new location is estimated at $4000. The economic benefit of this grading is estimated at $10,000. (see following tabulation based on curve 1, Fig. 2, p. 13).

TABULATION OF CAPITALIZED MOTOR OPERATION COSTS
(Old and new locations, Stas. 0 to 12 + 00)

Rate of grade, per cent	Estimated capitalized cost, motor operation, 100 vehicles daily per foot of distance	Old profile		New profile	
		Distance feet	Amount	Distance feet	Amount
1 or less	$ 9.10	80	$ 728	50	$ 455
2.0	9.15	320	2,928		
2.6	9.21			200	1,842
3.0	9.25	100	925		
3.4	9.31			100	931
3.9	9.37			100	937
4.0	9.40	80	752		
4.2	9 46	300	2,838	156	1,475
5.0	9 70			550	5,335
8.0	12.50	320	4,000		
Totals.........		1,200	$12,171	1,156	$10,975

Net capitalized advantage of new line per 100 vehicles daily, $1200.
Net advantage 800 vehicles daily, $9600, say, $10,000.

"4. **Suitable Pavement Types (Based on Traffic Classification).**—This road is Class II traffic (see Sec. 2 of this report). For this volume and character of traffic either bituminous macadam or cement concrete will serve satisfactorily. For the special conditions prevailing, it is probable that bituminous macadam is the more rational type to choose, based on the following facts. This relocation is a short piece of Road 294, which is a bituminous-macadam construction; it connects at the north end (Sta. 0) with Road 474, which is bituminous macadam, and with Road 94, which is bituminous macadam to the west and will be reinforced cement concrete to the east. This relocation will have some deep fills which will probably settle for 2 or 3 years after construction, which is not a favorable condition for rigid-pavement construction. The existing macadam of the old road can be excavated and reused for subbase course of the new macadam construction which gives a distinct economic advantage to the use of this type.

"For record purposes comparative estimates have been prepared for both bituminous macadam and cement concrete.

"5. **Soils.**—The subgrade soil is a medium clay comparatively free from ground-water seepage.

"6. **Uniform Strength Design.**

Station to station	Soil	Grading conditions	Total pavement depths[1]	
			Macadam	Concrete
0–2	Medium clay	Cut and shallow fills	16	7
2–7	Medium clay	Fills over 3" deep	11	7
7–9	Medium clay	Cut and shallow fills	16	7

[1] These depths are based on Table 74 (p. 391) for the macadam design and on the minimum allowable depth of concrete pavements (p. 453).

"As further evidence of proper allowable depth of macadam on this road the existing macadam (10" deep) is in perfect shape on the high-fill Stas. 0 to 5 after 10 years' use. At no point on Road 294 has there been any evidence of weakness where the old macadam is 16" deep in cuts. This old road carries a regular milk-collecting truck route and this evidence extending over 10 years seems to strengthen the conclusions expressed by Table 74 (p. 391).

"For the macadam construction a 3″ depth of top course is recommended; a 4″ depth of new stone or slag for middle course; and from 4 to 9″ of reused old macadam for the foundation course.

"7. **Materials Required (Pavement Estimate).**—There are 1900 sq. yd. of pavement on this job. The macadam type of pavement will require: 320 cu. yd. foundation course (4 to 9″ thick); 200 cu. yd. middle course (4″ thick); 160 cu. yd. top course (3″ thick); and 4300 gal. of bituminous binder.

"The concrete type of pavement will require: 360 cu. yd. concrete 1 : 1½ : 3 mix (6⅜″ thick); 700 bbl. cement; 17,200 sq. ft. mesh reinforcement (40 lb. per 100 sq. ft.); 7000 lb. bar reinforcement; 400 lin. ft. expansion joints.

"8. **Materials Available.**—The only local material available is the existing macadam, Sta. 0 to 9, which can be excavated and reused for foundation course macadam construction; 350 cu. yd. of this old macadam are available for reuse.

"All other materials must be imported: cement (Pittsford delivery); stone (Pittsford delivery or truck from Rochester); sand (Pittsford delivery or truck from Rochester); bitumen (Pittsford delivery or Tarvia from Prighton); water (hydrant supply Stas. 0 and 9, Lake Ontario Water Company).

"9. **Comparative Estimates** (See Table 97, p. 520).

Type of pavement	Average thickness, inches	Cost per square yard of pavement				
		Estimate construction cost	Yearly interest	Yearly maintenance	Yearly renewal	Total yearly charge
Bituminous macadam......	13	$2.10	$0.105	$0.035	$0.11	$0.25
Cement concrete..........	7	3.20	0.160	0.010	0.12	0.29

"Grading, culverts, and incidentals are estimated at approximately $4900.

"The total cost, including all items with an allowance of approximately $800 for engineering, inspection, and contingency, is as follows:

Bituminous macadam..........................	$ 9,500
Cement concrete..............................	11,809

"10. **Maximum Allowable Expenditure and Final Recommendations.**—Any necessary expenditure is allowable on this project on the score of safety as outlined in the first paragraph of the report. The economic advantage of the new location due to better grades and shorter distance probably warrants a construction expenditure of about $10,900 to $11,000 (see Sec. 3 of this report). The new pavement does not reduce motor operation costs appreciably, as the present pavement is equivalent to a first-class asphaltic concrete surface as far as operation is concerned. The total economic value of the proposed work does not probably exceed $11,000 to $12,000.

Either cement concrete or bituminous macadam could be constructed without loss to the community, but the selection of bituminous macadam seems a better business investment in this particular case, as discussed in Sec. 4 of this report.

(*Signed*)
Designer."

TYPICAL REPORT ILLUSTRATING THE EFFECT OF UTILIZATION OF OLD MACADAM ON RECONSTRUCTION COSTS

Rochester, N. Y.,
Apr. 10, 1923.

"Mr. ————————,
Division Engineer,
Rochester, N. Y.

"Dear Sir:

"Find below summarized design report on the reconstruction of Road 9 State Route 30, Class IIA traffic, from Stas. 89 to 239 + 35, a distance of 2.83 miles.

"We recommend an entirely new cement-concrete pavement 7″ average depth, Stas. 89 to 132. From Stas. 132 to 239 we recommend that the old macadam (at present ranging in depth from 8 to 11″ as determined by 50 test holes) be retained as a base course, widened to 18′ with gravel foundation course, the thickness of the old road increased by the addition of a 4″ slag water-bound middle course, which is to be used by traffic at least 3 months, and then capped with 3″ of some type of asphalt concrete preferably Amiesite or Topeka. This design is estimated to cost $13 ,000. If cement concrete is used for the entire distance, the estimated cost is $146,000. These two designs are practically equal from the standpoint of maintenance and motor operation costs.

"An alignment change is made between Stas. 109 and 114; this change in alignment is amply justified on the score of safety regardless of the increased cost. It increases the cost over following the old road about $11,000, but it probably cheapens the cost of motor operation enough to warrant a construction expenditure of at least $6000, even if the increased safety is given no weight in the decision.

Sincerely yours,
Designer."

Actual Procedure.—This road was constructed of concrete for the entire length on account of impracticability of using both cement concrete and bituminous hot mix equipment on such a short job.

CHAPTER IX

THE PROTECTION AND ELIMINATION OF RAILROAD GRADE CROSSINGS

Introduction.—Railroad grade crossings are a source of danger and delay to highway traffic. Danger is present in all cases and is the main factor to be considered. Delay, with resultant congestion, becomes important on city streets or on heavily traveled rural roads which are frequently blocked by long, slow freight trains. Danger can be materially reduced by means of various kinds of warning and protective devices or entire safety secured by grade separations or highway relocations. Delay can only be corrected by grade separations or relocations.

It is well recognized that the railroads and community at large are obligated to provide adequate safeguards at these crossings to protect reasonably careful drivers. It is also quite well established that large expenditures for the complete protection of reckless road users are rarely justified, as such individuals are a positive menace to highway traffic and their death is a gain rather than a loss to the community. This general premise results in the use of warning signs and signals for the great majority of the crossings and the use of eliminations on the more heavily traveled highways where delay, congestion, or unusual physical conditions make such construction desirable for general community benefit.

It should be borne in mind however that while subway or overhead crossings eliminate danger of collision between trains and highway vehicles they introduce other sources of highway accident due to grades, curved alignment etc. That is unless such eliminations are well designed they are often the source of accidents in themselves which did not occur before the eliminations were constructed. (*See pages* 632, 34 *and* 646)

Cost and Relative Value of Different Kinds of Protection.—Adequate protection for careful highway users can be accomplished with comparatively small expenditures, which are well within the means of most communities, by the intelligent use of different kinds of warning signals and watchmen. The effectiveness of such a program, however, depends on recognition of the limitations of the different kinds of warning devices and the selection of the best type or combination of methods which will meet the special conditions at each crossing. The advantages and limitations of the different methods of grade-crossing protection are discussed under each type of signal. Complete protection and the prevention of delay by means of grade separations are costly matters, which even the richest communities find difficulty in financing for any large percentage of crossings. The following table indicates in a general way the cost of different types of protection on rural highways for western New York conditions in 1925.

TABLE 111.—TABLE COSTS OF DIFFERENT TYPES OF PROTECTION

Item	Approximate cost of installation	Yearly maintenance and operation	Yearly renewal	Total yearly cost interest 5%, maintenance and renewal
Approach warning signs...	$10	......	$ 2	$ 3
Pavement markings.....	....	......	15	15
Railroad signs..........	30	$ 3	5	10
Automatic signals, steam railway..............	1000–2000[a]	100	150	350[a]
	3000–6000[b]	200	300	700–1000[b]
Automatic signals, electric railways.......	2500	20	200	350
Automatic signals plus 8-hr. flagman..........	.	.	.....	1400
Flagmen (24 hr.)........	100	3100	.....	3100
Hand gates...........	400	3200	100	3300
Electric gates with tower	2000	3500	200	3700
Grade-separation eliminations..........	80,000–200,000 100,000 average	200 ±	1300 ±	6500 ±

[a]Single and double track. Usual conditions.
[b]Four tracks.

The following table, based on accident data in Division 4, western New York, for a period of 10 years gives some basis for judgment as to the relative effectiveness of signal and watchman protection (see Table No. 112).

TABLE 112

Method of protection	Relative number of accidents for uniform density of traffic for different methods. Corrected for traffic volume to make directly comparable with each other	Reduction in danger accomplished over unprotected conditions, %	Remarks
Unprotected or poorly protected crossings....	10	..	Good data
Plain semiphore or banjo semiphore type of automatic signal..........	6	40	Meager data
Automatic wigwag oscillating-arm signal with bell attachment.......	3	70	Fair data
24-hr. watchmen or gates	2	80	Good data
Stop orders on electric interurban cars........	0.5	95	Good data

NOTE.—Automatic intermittent flash signals have not been in use long enough for the accident records to be of much value. They are apparently about comparable with the wigwag signal in effectiveness, but are not yet as well understood by drivers as the swinging disc. This difficulty will be rapidly overcome with a little longer use. Where flash signals face directly into a low strong sun they are ineffective while this condition prevails.

A very conservative estimate of the relative value of these methods of protection assumes that automatic signals reduce danger from 40 to 60%; watchmen and gates from 50 to 80%; and well-designed eliminations 100% plus the added advantage of the free passage at all times. It can be readily seen that the greatest reduction in danger for the expenditure of limited funds can be obtained by the use of automatic signals in conjunction with well-built approaches, warning signs, and pavement markings for the ordinary rural highway crossing. It can also be seen that where flagmen and gates are needed on account of heavy traffic in conjunction with railroad switching and shifting, which cannot be adequately served by automatic signals, grade-separation eliminations are a good business investment, provided the topography permits their erection for a reasonable first cost and the element of traffic delay adds to their value. Figure 192 shows cost and economic limit of different methods of protection for different traffic densities in western New York (page 602).

A common-sense solution which has been widely adopted is to erect automatic signals supplemented by watchmen where needed at all crossings on important roads and to proceed with a restricted program of elimination construction as the funds become available.

Financing.—The percentage of yearly highway funds that can be properly used for crossing protection should have some relation to crossing danger as compared with the other sources of highway danger. The division of cost between the railroads and the community and the method of taxation for public funds should have some relation to the benefit derived from such protection.

While the relation of railroad-crossing accidents to other highway accidents will vary widely for different localities, available statistics given in Chap. I (p. 34) indicate that, as a rule, grade crossings are responsible for from 6 to 10% of all highway accidents. Taking the country as a whole, they are responsible for two deaths per year per 100,000 population. On the state highway system in western New York, involving 207 grade crossings and an average highway traffic of 800 vehicles daily, there have been approximately 190 serious accidents in 10 years, or an average of about 0.1 serious accident per crossing per year. From the standpoint of damage claims, assuming all accidents, whether due to carelessness or not, the average money damage would not exceed $300,000 total, or $1500 per crossing per year average for this territory, and probably did not actually exceed $1000 per crossing. This indicates, in a general way, that there is very little basis for using a large percentage of highway funds for crossing protection; that 10% would be an extreme maximum; that 5% would be a large percentage; and that 2 to 3% is generally a more rational figure, considering that this phase of highway construction adds little to the economic value of the system. The railroads are generally willing to spend somewhat more than their claim accounts for their share of such work.

It is quite generally recognized that, from a practical standpoint, it is desirable to collect the necessary funds from both railroads and the local communities with some state aid. Whatever the railroad pays is, of course, finally collected from the community

(text continued on page 603.)

Recommended Methods of Protection

1. Where crossings are located between stations and trains are travelling at normal speed and no objectionable delay occurs we recommend good type automatic signals.

2. Where crossings are near station stops or yard switching on medium traffic highway we recommend automatic signals supplemented by part or full time flagmen.

3. Heavy traffic rail and highway with congestion and delay requires eliminations.

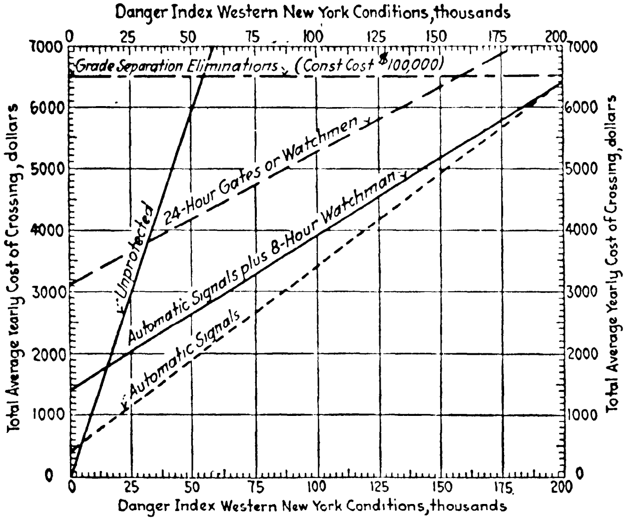


Fig. 192.—Diagram of probable total average yearly cost of crossings for different volumes of traffic protected by different methods.

Note: Total cost for each method of protection includes cost of protection plus probable accident damage figured at $15,000 for each serious individual injury. Diagram based on 10 year accident data on 200 crossings considering different volumes of highway and railroad traffic, visibility and physical condition of approaches.

Economic Limits for Different Methods of Protection

No protection—maximum limit— danger index 4000

Good automatic signals { min. limit—danger index 4000; max. limit—danger index 200,000

Signals + part time watchman { min limit—danger index 15,000; max. limit—danger index 200,000

24 hour watchmen or gates { min. limit—danger index 30,000; max. limit—danger index 160,000

Grade separation elim.—Min. limit danger index 40,000 to 100,000 depending on const. cost.

From a practical standpoint the minimum limits are generally reduced by from 20 % to 50 % below these figures as crossing protection cannot be considered entirely, from a business point of view but the railroads should not assume the added cost over and above the

(continued at bottom of page 603.)

through freights and rail charges of all kinds and beomes a general community charge. If the amount paid by the railroad for these protective measures does not exceed their yearly damage-claim bills for unprotected conditions, their operation charge is not increased and the general public pays no special extra amount for such protection. If the amount exceeds the damage-claim account, it eventually becomes a general community charge The direct tax for public funds can be collected as a general tax or as a special vehicle tax.

The railroads are benefited by protective measures in proportion to the reduction of their damage-claim amounts. The highway user is directly benefited by added safety of travel to him personally. The community at large derives only a small economic benefit from such work unless it results in the elimination of delay on heavily traveled roads. It is a well-established principle that both railroads and the public should cooperate in the cost and operation of protective measures to the extent to which they are directly benefited and, while the principle is rarely adhered to exactly, it is desirable to hold as closely to it as practicable. This indicates, in a general way, that public funds should be raised by vehicle taxation for this class of work, but should probably not usually exceed 5% of the total highway funds, and that the railroad funds should probably not greatly exceed the damage-claim accounts.

Following the general principles of highway finance outlined in Chap. 1 (p. 19), all expenditures for temporary signals, watchmen, etc., should be paid from current yearly funds, while the more permanent elimination construction may well be financed by 40-year serial bonds.

Sample Report for a Specific General Case.—To illustrate the practical application of the foregoing discussion, conditions in Division 4, western New York state highway system will be cited. The figures given are approximate only, but are sufficiently accurate to warrant general conclusions for quite typical conditions.

"1923 BASIC DATA, DIVISION 4, NEW YORK STATE, RAILROAD-CROSSING PROGRAM

Area	4.500 sq. miles
Population	550,000
Assessed valuation	$600,000,000
Motor registration	100,000
Total road mileage (rural)	6,700
State system mileage (proposed)	1,350 ±

Notes for Fig. 192 continued from page 602

economic limit for the method of protection ordered by state or national governments.

Probable number of serious accidents per year
= danger index ÷ constant "P"

Values of constant P

P = 125,000 unprotected crossings
P = 500,000 wigwag or flash signals
P = 700,000 24 hour watchmen and gates

Note: For explanation of danger index see text of report, page 623.

Complied by
W. G. Harger, 1925
Grade Crossing Eng.

Average daily volume of highway traffic per mile of road	900
Percentage of total vehicle miles carried by state system	40% ±
Replacement value of finally complete state system, pavements, and bridges	$60,000,000 ±
Estimate yearly tax budget on completion of system, including maintenance, renewal, and construction-bond charges (pavements and bridges)	$4,000,000 ±
Total number of railroad crossings on state system (steam and electric)	251
Number of grade separations existing	44
Number of grade crossings well protected	57
Number of grade crossings poor or no protection	150
Number of miles of state road per crossing	5.4
Total average number of trains per day passing all grade crossings	6,000 ±
Total average number of highway vehicles per day passing all crossings	180,000 ±
Total danger index[1] (average daily potential risk) at grade crossings	4,000,000
Number of serious injuries per year (10-year average)	20 ±
Estimated damage, including personal-injury claims and minor accident damages	$250,000 per year
Serious injuries per 100,000 population per year	3.6
Daily danger index for single crossings representing probability of one serious accident per year (based on 10-year period, fairly good data):	
a. Unprotected or poorly protected crossings	150,000
b. Well-protected (watchmen or signals)	700,000
c. Stop orders, electric cars	No accidents occurred
d. Average all grade crossings in division	200,000

"The following data has been kept for a short time and on only a few signals, and must be used with caution:

e. Wigwag automatic signals	500,000
f. Banjo semiphore signals	250,000

"This indicates that signals and watchmen decrease danger from 50 to 70% over unprotected conditions and that stop orders on electric interurbans are effective in reducing danger. It also shows that, to be most effective, an automatic signal should be of the oscillating or intermittent-flash type to catch the eye readliy. *During 1926, grade-crossing accidents were only 70% of the 1924–1925 rates, due to the installation of 37 automatic signals in 1925 and 1926 at the most dangerous unprotected crossings.*

"The local engineers recommended the following program for this district as a reasonable solution, for 1923 conditions considering the conditions tabulated and the general principles of protection and financing previously outlined."

"Summarized Conclusions and Recommendations, Division 4.—It is not likely that it will be possible to obtain more than $100,000 to $150,000 per year for the state's and counties' share of work to reduce danger at grade crossings on the state road system in this division. After a careful study of conditions in the division and discussions with county officials and automobile owners we recommend the following program as being an effective way to reduce danger with the funds at our disposal.

"1. The immediate installation of a uniform type of automatic audible and visible signal, preferably of the flash type (supplemented by watchmen in the villages), at all unprotected crossings on steam railroads, and either signals or stop orders on electric interurban roads. This work will cost about $200,000 and should be completed in 1925–1926. This work can be expedited by a modification in the railroad law to permit state cooperation in the cost of installation of such signals at state road crossings. It is certain that this work will result in more total reduction in danger to careful motorists than any other method of expending the first $200,000. This part of the program also includes bringing the physical highway conditions at the crossing and on the approaches up to the requirements of the Standard Specifications for grade crossings given on page 609.

"2. A 10-year program for the elimination of about 50 of the most dangerous grade crossings in the division estimated to cost $5,000,000 financed by 40-year serial bonds. The yearly budget for their retirement to be financed

[1] **NOTE.—For derivation of danger index see page 623.**

by motor-vehicle taxation for the state and county share of cost (50%) and railroad funds for the railroad share (50%). The state and county yearly budget for this work in this division will amount to about $125,000 yearly.

"3. The retention of signal protection at the relatively unimportant crossings until future necessity requires their elimination at an estimated cost of about $15,000,000.

"NOTE.—Existing eliminations, watchmen, signals, etc., have already eliminated at least 40% of the danger on the state highways in this division (steam and electric railroads).

"It is estimated that the remaining existing danger in the division at grade crossings will be reduced at least 50% for careful drivers by the expenditure of the first $200,000 for signals and that at least 90% of danger over the division as a whole will be eliminated on the completion of the 50 most important grade-separation projects with the balance of the crossings protected by automatic signals or watchmen.

(*Signed*)
W. G. Harger.
Grade Crossing Engineer."

Sample Report for a Specific Individual Crossing.—In order to make a reasonable analysis and arrive at any rational conclusion in regard to the best method of protection for any specific case considering existing danger and funds available, it is necessary to consider probability of danger (see p. 623), probable yearly damage claims (see p. 602), and the relative effectiveness and cost of different methods of protection (see p. 602). A sample report follows illustrating the practical application of the data referred to.

"REPORT ON CROSSING 8, DIVISION 4, NEW YORK STATE HIGHWAY DEPARTMENT

County of Monroe, Division 4
Highway 62, P. S. Case
Name of crossing, West Henrietta Road
Name of highway, West Henrietta-Rochester
Name of railroad, Lehigh Valley
Branch, Rochester to Rochester Junction
Town of Henrietta
Number of tracks, 1
Type of ballast, Gravel
Weight of rail, 85 lb.
Alignment of railroad, tangent
Approximate railroad grade, level
Any contemplated railroad improvement, none
Number of passenger trains daily, 12
Speed of passenger trains, 40 m.p.h.
Number of freight trains, 4
Speed of freights, 20 m.p.h.
Protection at crossing, none
Angle between center-line road and track, 58°
Highway pavement, bituminous macadam 16′ wide.
Pavement between rails, bituminous macadam 20′ wide
Highway approach grades, 1.22% north, 1.5% south
12-hr. highway traffic count (August), 2,500
Estimated average daily 24 hr. (year round), 2,200
Visibility of approaching trains:
  Poor in two directions
  Good in two directions
Accident record, four accidents in 10 years, resulting in four deaths and two serious injuries.
Probable yearly damage claims, average, $9,000 ±
Most feasible type of elimination, overhead highway.
Estimated cost of elimination, $110,000
Most feasible method of temporary protection, automatic signals

"**Recommendations.**—We recommend the immediate installation of automatic signals at this crossing and its elimination as soon as funds become available. We recommend that the pavement between the tracks be widened to 30′ and that the approach pavement be widened to 24′. While an elimination is undoubtedly justified, funds are not at present available and the signals will pay for themselves in a very short time and reduce danger at least 60%. These recommendations are based on the following data:

"**Danger Index.**

"Highway traffic factor	2,200
Visibility factor	1.2
Approach factor	1.0
Train factor	15

"Danger index in 1923 = 2,200 × 15 × 1.2 = 39,600, say, 40,000
Estimated index 10-year period 48,000, allowing for increase in volume.

"Probability of accident over next 10-year period for different methods of protection.

Unprotected $\frac{48,000}{125,000}$ = 0.4 serious accidents per year.

Automatic signals $\frac{48,000}{500,000}$ = 0.1 serious accidents per year.

24-hr. watchmen $\frac{48,000}{700,000}$ = 0.07 serious accidents per year.

Elimination = no accidents.

"Probability of yearly claim bill for different methods of protection, allowing $15,000 per serious injury.

Unprotected	$6000
Automatic signals	1500
24-hr. watchmen	1000
Elimination	None
The yearly cost of automatic signals is about	$ 400 ±
Watchmen	3000 ±
Elimination	6500 ±

"The use of signals will probably reduce danger about 70% and result in a net saving at this crossing of about $4000 per year. Watchmen will result in a net saving of about $2000 per year. Elimination will cost about the same as at present, but eliminates danger and delay. This crossing is number 11 in order of importance in Division 4 for elimination construction, and is number 2 in order of importance of the Lehigh Valley crossings.

(*Signed*)
Grade Crossing Engineer."

Causes of Accidents and Protective Measures.—A large percentage of the accidents, probably at least 50%, are caused by reckless driving, and no system of signs or signals will entirely eliminate such injuries (see Accident Causes, Chap. I, p. 33).

Accidents not directly attributable to recklessness are usually due to the following conditions and can be remedied at a small cost.

1. Motorists do not realize that they are approaching a crossing. This danger is reduced by proper approach warning signs (Fig. 196), pavement markings (Fig. 198), and standard crossing signs (Fig. 199).

2. Motorists do not realize that trains are approaching the crossing. This danger is reduced by automatic signals or watchmen and gates (Figs. 200 to 202). As a considerable percentage (15 to 20%) of collisions are due to automobiles running into the sides of freights trains at night, during rain or snow storms, it is necessary that signals or gates be placed on both sides of the tracks at the crossing. Accident records show the undesirability of central obstructions of all kinds on narrow rural highways, which makes it

desirable to locate the signal standards outside of the pavement area (see Fig. 193). In a similar manner gates should be located far enough back from rails to permit a standing automobile between gates and track in case a car gets caught inside of the lowered gates.

View of approaching trains can often be materially bettered by cutting trees, moving buildings and any other obstruction to view along the tracks. For a discussion of visibility see page 624.

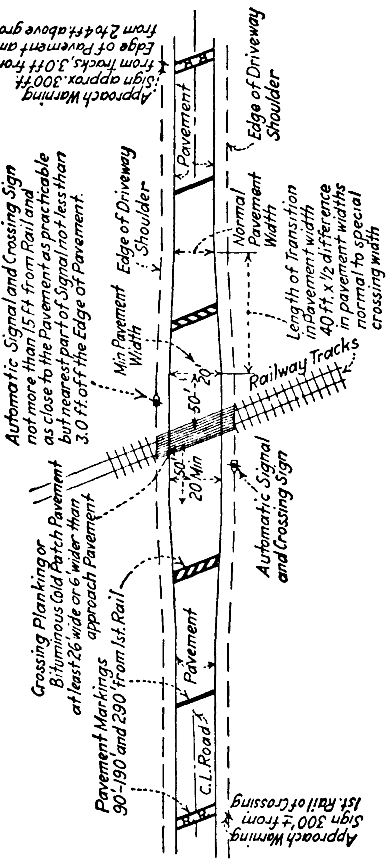


FIG. 193.—Typical rural highway grade crossing layout for signs, signals and pavement and planking widths.

3. Motorists get one wheel of their car off the pavement proper and hit the railroad track, which results in stalling the engine or diverting the machine from the roadway along the track or into a ditch. This can be remedied by making the planking or pavement between the rails wider than the approach pavement (see Fig. 195).

4. Cars often stall on the tracks due to inexperience and nervousness of drivers in applying power over a rough crossing. The remedy for this condition is to make the crossing smooth and keep the rails in the same plane, so that cars will be kept in motion with

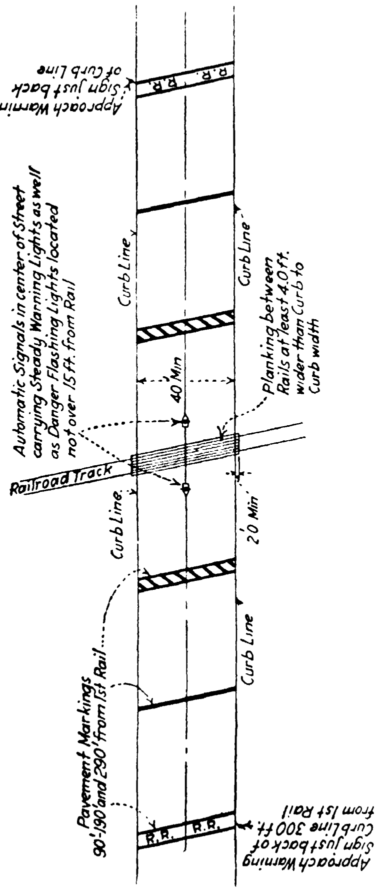


FIG. 194.—Typical village or city street grade crossing layout for signs and signals. *Note:* Central location of signals not permissible for widths of less than 40 ft. curb to curb. For less width use side location as shown in Fig. 193 for rural layouts.

a minimum of power. Where more than one track is crossed, the rails should all be at the same elevation on railroad tangents, and on curves the railroad track superelevation work on adjacent tracks should be sufficiently close to the same plane to eliminate noticeable bumps.

5. Trains are derailed by collisions with motor vehicles resulting in injury to passengers or train hands. This type of accident accounts for 1% of injury and death at crossings according to Interstate Commerce Commission 1925 Statistics.

The Hoover National Committee on Highway Safety states, as a basic principle of grade-crossing protection, that ease of motor operation is essential and that artificial obstructions, bumps, or jogs in line of travel should be avoided on the score that they have proved to be ineffective in reducing speed and derailment accidents, and that they distract the driver's attention, which should be focused on looking for approaching trains.

General specifications for satisfactory grade crossings follow and are illustrated by Fig. 193 and 194.

SPECIFICATIONS FOR RAILROAD GRADE CROSSINGS

"**Signs and Signals.**—Uniform type of reliable audible and visible signals located not more than 15′ from track, and about 4′ outside of the pavement area on the right-hand side of traffic; height of signal to be adjusted for local conditions and type of signal. Two signals to be erected, one on each side of crossing; these signals to be supplemented by watchmen for at least 12 hr. in villages and for at least 16 hr. daily on roads carrying over 3000 vehicles daily, where switching or station stops occur close to the crossing. The visible part of such signals to be of an oscillating or intermittent-flash type. All signals to be equipped with an automatic "Out of Order" sign properly illuminated if anything goes wrong with the mechanism.

"Standard crossing signs to be located on signal standard and illuminated at night by a steady hooded light to prevent glare into eyes of approaching drivers.

"Standard disc warning approach sign 300′ from crossing outside of pavement area on right-hand side of traffic line (not more than 3½ nor less than 2′ above grade of highway.

"Standard pavement markings.

"Special checkerboard warning sign 500′ from crossing if conditions warrant this extra sign.

"Highway traffic Stop Order signs within 50′ of rail enforced by police regulations for exceptional cases.

"**Pavement Width.**—At least 6′ wider between rails than on approaches. At least 20′ for 50′ each side of rails.

"**Highway Alignment.**—Straight alignment at crossing and for at least 75′ each side of crossing.

"Minimum radius of curvature on approaches 500′. Minimum acute angle between road and track center line preferably not less than 60 degrees.

"**Highway Grades.**—Maximum of 3 % for 75′ each side of crossing.

"Maximum downhill approach grade, 6 % with 200′ vertical-curve approach or 7 % with 300′ vertical approach.

"Maximum upgrade approach, 8 % with 200′ vertical-curve approach.

"**Sight Distance along Highway.**—Three hundred feet minimum.

"**Railroad Track.**—Where tracks are on tangent, all rails to be at same elevations. Where more than one track exists and the rails are superelevated for curvature the track elevations must be adjusted so that no severe bumps occur, which would tend to stall an automobile engine in high gear at low speed (or the top of rails should be in as near the same plane as possible).

"See Fig. 193 for typical grade-crossing approach."

Pavement or Planking between Rails.—The pavement between the rails of the track must be wide enough to prevent the wheels of highway vehicles from running off the edge and striking the rail, with resultant stalling of the vehicle or its diversion from its normal line of travel along the highway. This is particularly important on skew-angle crossings.

Where the approach pavement is a curbed street, 2′ additional width outside of the curb on each side for the pavement or planking between rails should be sufficient. On rural highways with earth shoulders, the rail planking should be made wider than the outside limits of the earth shoulder on which it is safe to drive. On a well-traveled improved state road this usually requires 27 to 30′ width at right angles to the road center line.

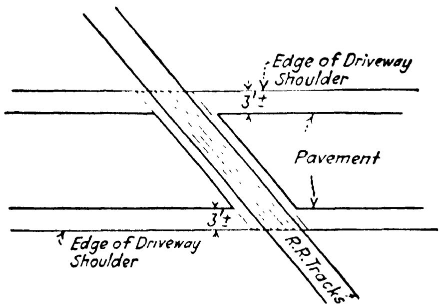


FIG. 195.—Planking between rails at crossing.

Well-kept planking or rails makes the most satisfactory crossing, but a cold-patch bituminous pavement carefully maintained serves well if it has a firm, solid, grooved header along the rail gage which can be easily obtained by the utilization of old rails as per sketch. It is important that the paving between rails be a straight plane between top of rails. It is a common fault to curve it up, which makes a bad bump.

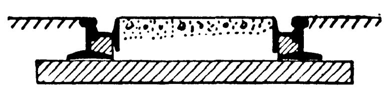


FIG. 195*A*.—Ordinary cold patch pavement between rails with rail gruved header. Satisfactory if surface of cold patch is kept in a smooth level plane between rails.

Types of Signs and Signals. Approach Warning Signs.—Any well-placed legible sign of distinctive and uniform shape and legend, so that it readily catches the eye, and its shape as well as the legend indicates railroad crossing, will serve the purpose. National uniformity in regard to shape, legend, and location is desirable.

These signs are usually placed about 300′ away from the track and are located on the right-hand side of the pavement at an elevation easily picked up by the eye and lighted at night by the usual car headlight 2 to 3 ft. above the ground. See also page 687.

Figure 196 shows two popular types; the disc type, and the checkerboard type. Figure 197 shows the usual location of such signs, which is 4′ off the pavement edge on rural roads and as close to the curb as possible without an overhang of the sign into the street on curbed city or village streets. The sign is usually set at an angle of about 15° to improve visibility as the driver nears it. These signs cost about $10 apiece to install, and have a maintenance and renewal charge of about $2 per year.

Pavement Markings.—Pavement approach warning markings are intended as an additional safeguard, particularly at night, in case the driver fails to see the approach warning signs at the side of the road. Various legends and systems of stripes are used. Figure 198 shows a useful design which serves traffic well. National uniformity is desirable.

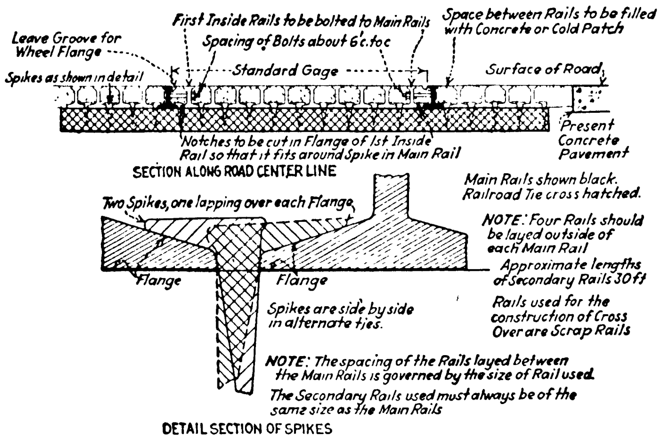


FIG. 195*B*.—A very excellent type of crossover using old rails surrounded with either cold patch or cement concrete.

These markings are made with the usual pavement-marking paint. On the usual road, one application a season will be enough. On heavy-traffic roads, two to three applications are required.

The average cost for paint and labor per year per crossing is about $15 (1925 cost conditions).

Standard Crossing Signs.—Standard crossing signs vary for different railroads and different localities. The eastern states favor the cross or diamond sign and some localities favor the circular sign. National uniformity is desirable and should be arranged.

The essential features of these signs should be large size, good location outside of the pavement area, and the use of two signs one on each side of the track as noted under the second cause of accident on page 606.

If automatic signals are used at the crossing, the standard crossing sign should be located on the same standard as the signal in order not to distract the driver's attention from the signal operation by two separate signs in different locations. All standard crossing signs should be illuminated at night by a steady hooded white or yellow light which does not throw a glare into the eyes of approaching drivers.

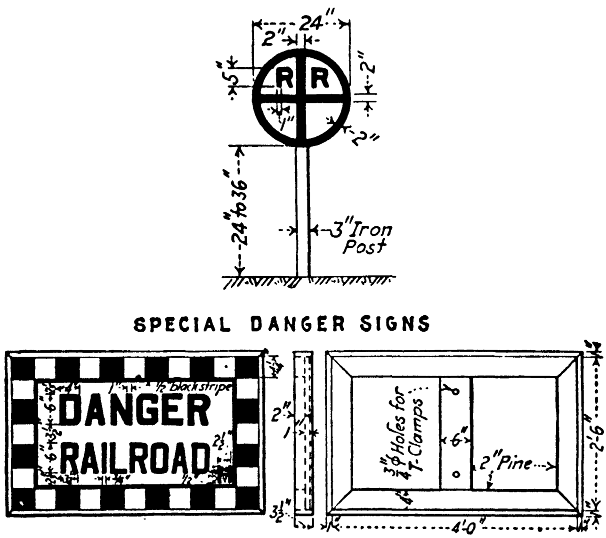


FIG. 196.—Typical approach warning signs.

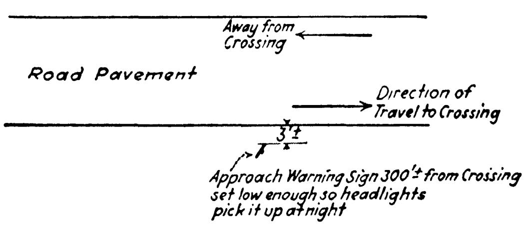


FIG. 197.—Location of approach warning signs.

Automatic Signals and Watchmen. *Limitations of Use.*—Automatic signals are primarily suited for the protection of crossings located between railroad station stops where trains are traveling at normal schedule speed. Under these conditions the signal warning has a short duration and always means that danger is actually imminent, and under these conditions traffic respects the signal and gives it due consideration. Where automatic signals are used near station stops, or where yard or siding tracks are used for considerable switching, the signal indicates danger for long periods when no danger actually exists. This tends to discredit the warning with the traveling public and they give it little attention, which reduces the effectiveness of the signal not only at the crossing in question but tends to discredit similar signals in proper locations. For such locations the automatic signal should be supple-

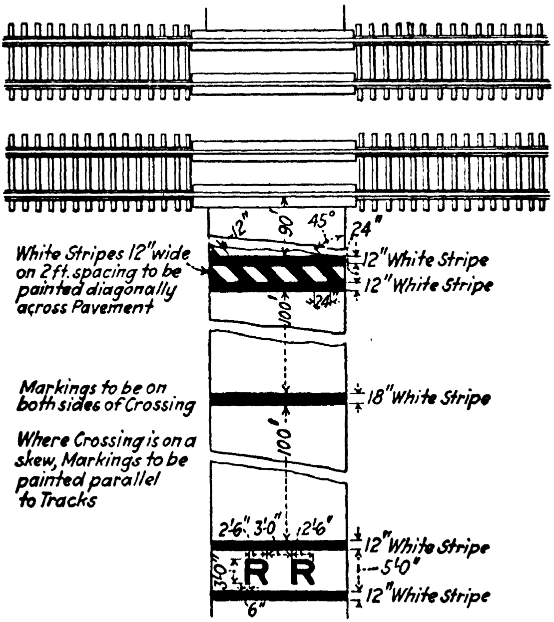


FIG. 198.—Pavement markings at railroad crossings. (N. Y. State, 1926.)

mented by flagmen during the portion of the day when the signal operation is made indefinite by long stops or switching.

Watchmen are particularly suited for use on heavily traveled highways where railroad trains either stop, switch, or shift continuously across the highway. The watchman can use judgment in moving traffic, and can safely speed up the movement of crossing traffic which reduces needless delay nuisance.

Conditions governing the proper use of signals or watchmen may be summarized as follows:

Condition	Recommended Method
Crossing between stops with trains traveling at normal schedule speeds.	Automatic signals.
Crossings near stations where stops or shifting occur for a large part of the time.	Automatic signals, supplemented by watchman part time or full time, depending on stop and switching schedule and volume of highway traffic.

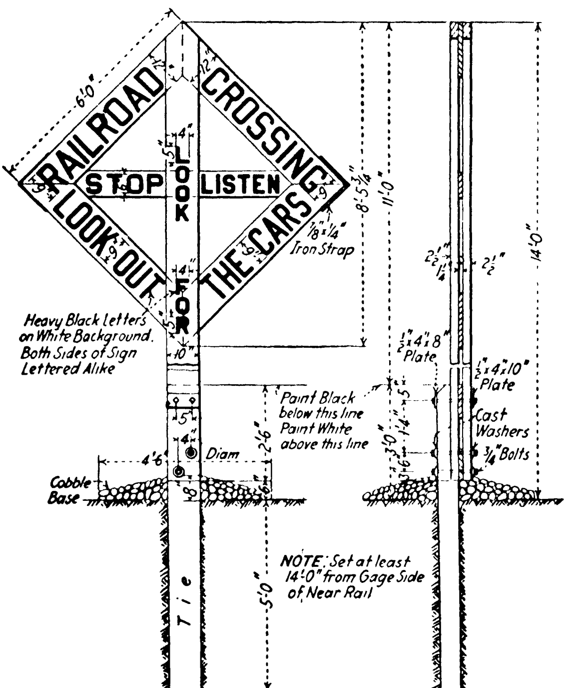


FIG. 199.—Standard diamond railroad crossing sign. Buffalo-Rochester and Pittsburg Ry.

Reliability of Protective Devices.—Mechanical devices have been perfected so that they are very reliable. All approved installations also automatically fall into the danger position if anything goes wrong with the circuits or mechanism.

Watchmen are subject to the human element of failure due to sickness or inattention. On heavily traveled crossings it has been found desirable to supplement the watchman with automatic signals to provide against this contingency. Under these conditions the automatic signal generally has a bell attachment to act as signal to the watchman as well as to the public.

Causes of Failure in Effectiveness of Protective Measures.— A common cause of accident is a wilful disregard of the warning on the part of the highway user, and there is no means of correcting this abuse except by police control.

The other causes are failure of signal or watchman to prevent accidents chargeable to failure of signal to function, too short a warning period, failure of driver to see or hear signal, failure of driver to understand signal, collision of road vehicle with signal posts or gates, and failure of signal to function due to unusual direction of train movements.

Failure of signal or watchman to function is a rare occurrence but does happen due to mechanical or human uncertainties. On heavy-traffic crossings, this can be overcome by the use of both automatic signals and watchmen, thus reducing the probability of failure to a very rare occurrence. Too short a warning period can be remedied by longer track circuits, giving about 25-sec. warning for the fastest trains on the line.

Failure of driver to understand signal is rare but occurs with sufficient regularity to warrant protection. It is largely due to non-uniformity of signal signs or lights, and a conflict between city street signals and rural railway-crossing signals. Gates or the oscillating disc and light signals are easily understood. The intermittent flashing light is sometimes confused with city isle-of-safety lights. It is suggested that the intermittent flashing red light be used only at railroad crossings to indicate imminent danger and that all isle-of-safety lights be steady yellow.

Failure to hear or see signal is due to type and location. Bell or hearing protection is obsolete, except as a minor adjunct, as motor traffic with closed cars and high speed gets little benefit from sound warning. The eye of a driver is more easily caught by a swinging disc or light or by an intermittent flash than by a steady light or steady semiphore arm in a danger position. The location of lights or oscillating discs must be well within the normal field of vision of the driver, which means as low as practicable without being obscured by rigs ahead of the driver, and as near the center of field of view as possible without causing an obstruction to the line of travel. These facts indicate that the most effective types of signal are the oscillating or intermittent-flash type supplemented by a bell attachment (see p. 618) located not more than 4′ off the edge of the pavement for rural conditions, the standards and crossing signs to be large enough and so painted that they catch the eye easily, the lights or swinging discs to be as low as practicable (see Fig. 201 and 202).

Collision between vehicles and stationary obstructions is quite frequent (see p. 35). Accident records in regard to central obstructions indicate the necessity of locating signal standards outside of

(text continued on page 620.)

TABLE 113.—SPEED AND DISTANCE TABLE[1]

Miles per hour	Time per mile: Minutes	Time per mile: Seconds	Feet per minute	Miles per hour	Time per mile: Minutes	Time per mile: Seconds	Feet per minute	Miles per hour	Time per mile: Minutes	Time per mile: Seconds	Feet per minute
100.00	0	36	8800	45.57	1	19	4013	29.50	2	02	2596
97.30	0	37	8562	45.00	1	20	3960	29.03	2	04	2552
94.74	0	38	8337	44.44	1	21	3911	28.57	2	06	2515
92.31	0	39	8123	43.90	1	22	3863	28.12	2	08	2474
90.00	0	40	7920	43.37	1	23	3811	27.69	2	10	2438
87.80	0	41	7726	42.86	1	24	3770	27.27	2	12	2400
85.71	0	42	7543	42.35	1	25	3724	26.87	2	14	2365
83.72	0	43	7368	41.86	1	26	3682	26.47	2	16	2330
81.82	0	44	7200	41.38	1	27	3641	26.09	2	18	2296
80.00	0	45	7040	40.91	1	28	3600	25.71	2	20	2262
78.26	0	46	6887	40.45	1	29	3559	25.35	2	22	2231
76.59	0	47	6741	40.00	1	30	3520	25.00	2	24	2200
75.00	0	48	6600	39.56	1	31	3480	24.66	2	26	2172
73.47	0	49	6468	39.13	1	32	3442	24.32	2	28	2138
72.00	0	50	6336	38.71	1	33	3406	24.00	2	30	2112
70.59	0	51	6212	38.29	1	34	3369	23.68	2	32	2085
69.23	0	52	6090	37.89	1	35	3334	23.38	2	34	2058
67.92	0	53	5973	37.50	1	36	3300	23.08	2	36	2032
66.66	0	54	5870	37.11	1	37	3265	22.78	2	38	2006
65.45	0	55	5775	36.73	1	38	3228	22.50	2	40	1980
64.29	0	56	5658	36.36	1	39	3203	22.22	2	42	1956
63.16	0	57	5562	36.00	1	40	3168	21.95	2	44	1934
62.07	0	58	5465	35.64	1	41	3134	21.69	2	46	1909
61.02	0	59	5368	35.29	1	42	3106	21.43	2	48	1884
60.00	1	00	5280	34.95	1	43	3078	21.17	2	50	1863
59.02	1	01	5192	34.61	1	44	3044	20.98	2	52	1841
58.06	1	02	5113	34.28	1	45	3017	20.70	2	54	1822
57.14	1	03	5025	33.96	1	46	2990	20.45	2	56	1800
56.25	1	04	4950	33.64	1	47	2955	20.22	2	58	1779
55.38	1	05	4875	33.33	1	48	2930	20.00	3	00	1760
54.55	1	06	4798	33.03	1	49	2904	18.46	3	15	1628
53.73	1	07	4720	32.73	1	50	2878	17.14	3	30	1506
52.92	1	08	4655	32.43	1	51	2851	16.00	3	45	1408
52.17	1	09	4590	32.14	1	52	2825	15.00	4	00	1320
51.43	1	10	4523	31.86	1	53	2807	12.00	5	00	1056
50.70	1	11	4462	31.58	1	54	2781	10.00	6	00	880
50.00	1	12	4400	31.30	1	55	2754	8.00	7	30	704
49.31	1	13	4338	31.04	1	56	2728	6.00	10	00	528
48.65	1	14	4279	30.77	1	57	2710	5.00	12	00	440
48.00	1	15	4224	30.51	1	58	2683	4.00	15	00	352
47.37	1	16	4165	30.25	1	59	2664	2.00	30	00	176
46.74	1	17	4107	30.00	2	00	2640	1.00	60	00	88
46.15	1	18	4061								

[1] WEISS, "Practical Railroad Maintenance," McGraw-Hill Book Company, Inc.

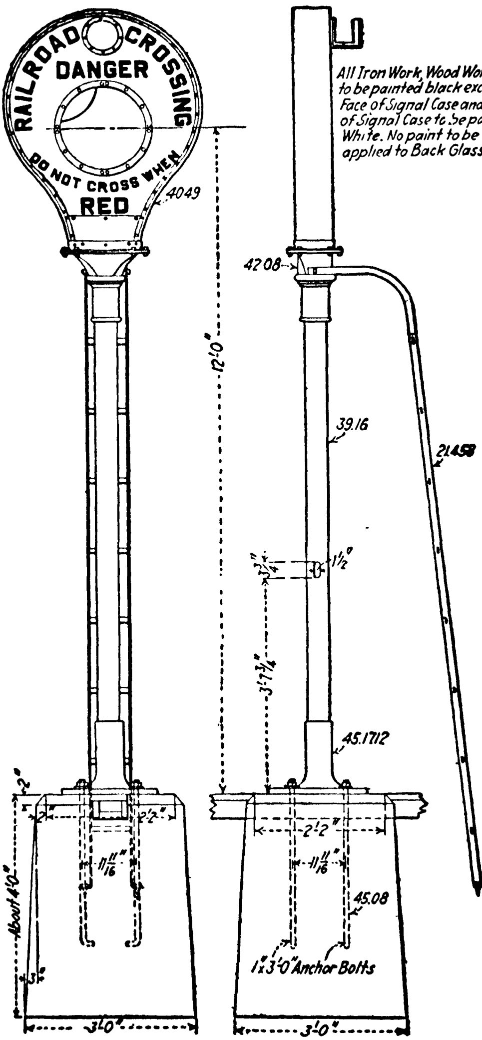


FIG. 200.—Banjo semiphore type of signal. (Lehigh Valley Ry. 1924.)

Note: This type of signal does not catch the eye so readily as the swinging disk or the automatic flash types and also 12 ft. is too high for easy vision particularly if auto has a sun hood.

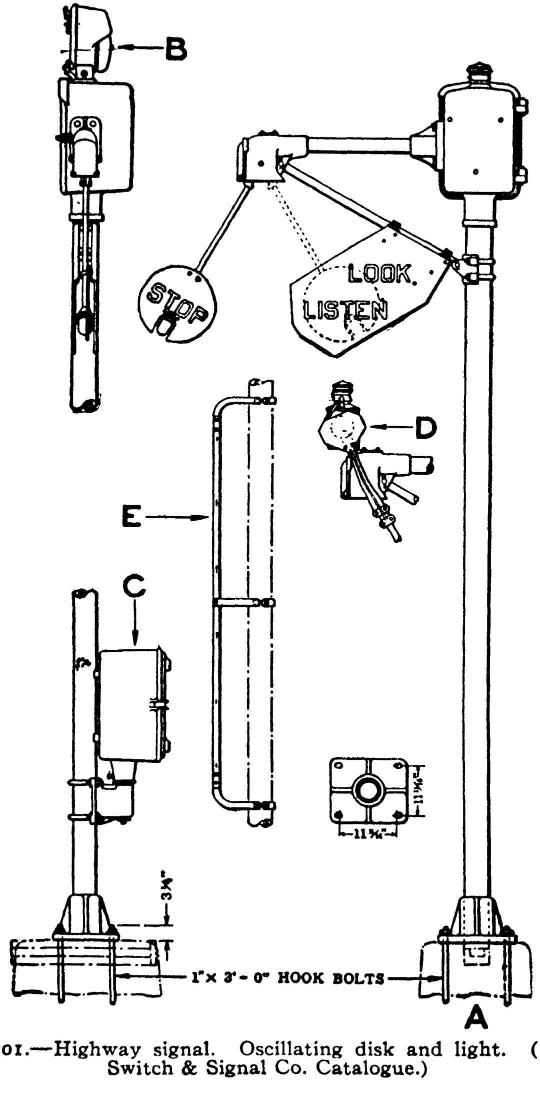


FIG. 201.—Highway signal. Oscillating disk and light. (Union Switch & Signal Co. Catalogue.)

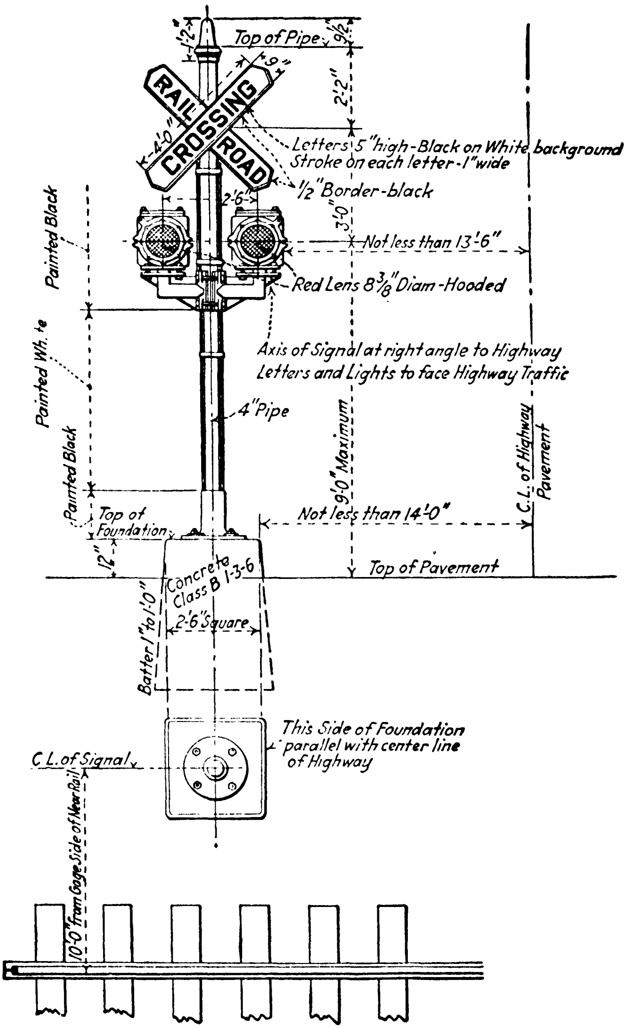


FIG. 202A.—Good typical design double flash signal for location at side of pavement rural highway crossings. (Buffalo-Rochester & Pittsburg Ry., 1926.)

the pavement area on rural highways. There is no question but what an automatic signal is more readily seen in the center of a road or street when it is actually flashing, but at night in rain or snow storms this location constitutes a positive menance when the lights are out and is the cause of enough accidents to nullify the visibility advantage of such a location. As a matter of fact, no cases are known in Division 4, western New York, where drivers failed to see the signals located on the side of the pavement. The central location also requires steady illumination, which is a bad

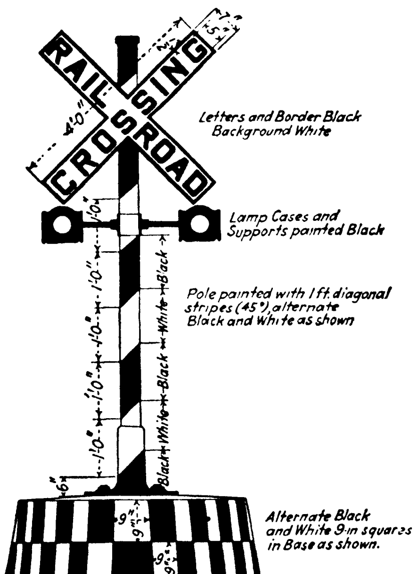


FIG. 202*B*.—Double intermittent flash signal with island base for location in center of streets. (Lehigh Valley Ry. Standards 1925.)

Note: Good design except that steady pilot lights are not shown.

feature during rain storms, as it blinds the eyes. The central location also adds about $1500 to $2000 to the cost of the installation for additional pavement approach widening. On wide village or city streets the central location is desirable, provided the standard carries steady warning lights as well as the flashers.

All automatic signals should be wired to take care of unexpected train movements in reverse direction to normal movement. This generally adds from $125 to $200 per installation on double-track roads, although in some cases it may add as much as $1000.

Types of Signals.—Bell signals alone are obsolete.

Plain, steady position semiphores (banjo type, Fig. 200) are fairly effective (see p. 617), but are going out of use in favor of the oscillating or intermittent-flash type.

Figure 201 shows a typical oscillating disc and light signal with bell attachment. These signals cost about $1300 to install per single signal, or about $2200 for two signals. Two signals should be erected at each crossing. The yearly maintenance is about $100 per year per signal. The signals are very effective (see p. 618).

Figure 202 shows a typical double horizontal intermittent-flash type of signal. These signals cost about $1500 for the erection of two signals (one on each side of track) on single- or double-track steam railroads, and about $2500 on electric railroads. The yearly maintenance is about $100 per year on steam roads and about $20 per year on electric roads. Where two or more highways intersect at the crossing, extra sets of lights should be provided facing each approach road. These signals are very effective and are the cheapest acceptable type to install and operate.

Specifications for these types of signal, covering all mechanical details of wiring, light intensity, lenses, and rapidity of flash or swing can be obtained from the American Railway Association, Signal Section, New York City. The essentials are as follows:

CODE PROPOSED BY SECTIONAL COMMITTEE OF THE AMERICAN ENGINEERING STANDARDS COMMITTEE

Section 4. Signals at Railroad Grade Crossings

"**Rule 40. Aspect.**—An electrically or mechanically operated signal used for the protection of highway traffic at railroad crossings shall present toward the highway when indicating the approach of a train the appearance of a horizontally swinging red light and (or) disc.

"NOTE.—This covers the use of so-called wigwags and of alternately flashing red lights, and the use of these devices should be restricted to the purpose of indicating the approach of a train.

"**Rule 41. Location.**—The railroad standard highway crossing sign and the signal shall be mounted on the same post.

"**Rule 42. Operating Time.**—Automatic signal devices for indicating the approach of trains shall be so arranged as to indicate for not less than 20 sec. before the arrival at the crossing of the fastest train operated on the track. Local conditions, such as three or more tracks, bad approaches, etc., should be allowed for by increasing the operating time, bearing in mind that too long an operation by slow trains is undesirable.

"**Rule 43. Flashing-light Type.**[1] *a. Height.*—The lamps should preferably be not less than 6′ nor more than 9′ above the surface of the highway.

"*b. Width.*—The two lamps shall be mounted horizontally, 2′ 6″ centers.

"*c. Flashes.*—Lights shall flash alternately. The number of flashes of each light per minute shall be 30 minimum, 45 maximum.

"*d. Hoods.*—Lamp units shall be properly hooded.

"*e. Range.*—When lamps are operated at normal voltage, the range, on tangent, shall be at least 300′ on a clear day, with a bright sun at or near the zenith.

"*f. Spread.*—The beam spread shall be not less than 3° each side of the axial beam under normal conditions. This beam spread is interpreted to refer to the point at the angle mentioned where the intensity of the beam is 50% of the axial beam under normal conditions.

"*g. Lenses or Roundels.*—The size shall be 5⅜″ minimum, 8⅜″ maximum.

"**Rule 44. Wigwag Type.**—*a. Length of stroke* is the length of cord which subtends the arc, determined by the center of the disc in its extreme positions, and shall be 2′ 6″.

[1] Good serviceable height, most cases 8′

b. Disc.—The disc shall be 20″ in diameter. Its field shall be white with a black circumferential border 1″ wide. The horizontal and vertical diameters shall be shown by black lines 2½″ wide. A red lens or roundel should be placed at the center in front of the lamp.

"*c. Number of Cycles.*—Movement from one extreme to the other and back constitutes a cycle. The number of cycles per minute shall be 30 minimum and 45 maximum.

"*d.* The lamp with which the disc shall be equipped shall be lighted when the disc is swinging.

"**Rule 45. Approach Signal.**—Advance warning signals which indicate approach to a railroad crossing, and not the actual approach of a train, shall conform to the provisions of sec. 3, Rule 30."

Section 5. Specifications for Colors

"**Rule 50. Definition of Colors.**—Red, yellow, green, or blue as used in this code are intended to mean:

"*a.* The colors resulting from the transmission of the proper light through the proper glasses and having the characteristics described in Rules 51 and 52.

"*b.* The colors resulting from the reflection of white light from the proper pigments and having the characteristics described in Rule 53.

"NOTE.—For a definition of white light, reference is made to the report of the Colorimetry Committee of the Optical Society of America, 1920–1921 (*Jour. Optical Soc. Amer.*, and *Rev. Sci. Inst.* 6, p. 563, 1922). Substantially, it is average sunlight at noon at latitude of Washington.

"**Rule 51. Qualitative Definition of Colors for Luminous Signals.**—*a. Red.*—The spectrum of red shall contain both red and orange but not more than a trace of yellow and no green, blue, or violet. The most desirable hue is entirely free from yellow, which means that the glass does not transmit the yellow light from a sodium flame.

"*b. Yellow.*—The spectrum of yellow shall contain red, yellow, and green, with but little blue and no violet. The most desirable hue is entirely free from blue and might be designated a light amber.

"*c. Green.*—The spectrum of green shall contain yellow, green, blue, and violet, with only a trace of red and orange. This hue is known as "admiralty green" and has a bluish tint when observed by daylight.

"**Rule 52. Quantitative Definition of Colors for Luminous Signals.**—The colors red, yellow, and green shall have the following characteristics:

	Dominant wave length, millimicrons	Purity, %	Integral transmission of glass, %
Red..........	Not less than 624	Not less than 100	Not less than 10
Yellow.......	Not less than 592 nor more than 600	Not less than 97	Not less than 24
Green........	Not less than 496 nor more than 536	Not less than 45	Not less than 11

"These values are determined by the transmission of light from a source at the color temperature of 2.360°K. (practically that of the acetylene flame or present type of vacuum tungsten lamp at normal voltage) through the respective glasses. They are based upon spectral transmission measurements and upon computations carried out in accordance with the methods and data described in the Colorimetry Report of the Optical Society of America.

"NOTE.—The light and dark limits of the glasses on which the above values are based have the following relative transmissions on the scale of the American Railway Association.

	Light Limit	Dark Limit
Red.........................	300	150
Yellow......................	200	100
Green.......................	250	100

"**Rule 53. Quantitative Definition of Colors for Non-luminous Signs.**—The colors red, yellow, green, and blue shall have the following characteristics:

	Dominant wave length, millimicrons	Purity, %	Integral of pigment, %
Red.........	Not less than 608	Not less than 60	Not less than 8
Yellow......	Not less than 580 nor more than 588	Not less than 80	Not less than 35
Green.......	Not less than 524 nor more than 552	Not less than 30	Not less than 8
Blue......	Not less than 466 nor more than 474	Not less than 10	Not less than 4

"These values are determined by the reflection of white light from the respective pigments. They are based upon spectral reflection measurements under conditions of diffuse illumination and upon computations carried out in accordance with the methods and data described in the Colorimetry Report of the Optical Society of America."

Probability of Accident. *Danger Index.*—Probability of accident depends on volume of highway traffic, number and speed of trains, delay and congestion at crossing, visibility at crossing, physical condition of highway approaches, and the method of protection used at the crossing. As a rough basis of arriving at a relative danger for a large number of crossings which it is desirable to rate in order of importance for a program of improvement the following method, used in western New York, is described.

Danger index, unprotected crossings = Average 24-hr. highway traffic × Weighed value, of number trains 24 hr. × Visibility factor × Approach danger factor.

To find the probability of the number of serious accidents per year (long-time average) the danger index for the crossing is divided by a special constant for each method of protection which represents one serious accident per year and is derived from a careful study of accident data for a large number of crossings for a number of years. Individual crossings will, of course, for short periods often vary considerably from these results, but in the long run the values are very reliable for this particular district. The tentative values for the constants used are as follows (these values are, of course, subject to future correction based on more complete data):

Unprotected crossings..............................	125,000
Plain semiphore or banjo semiphore....................	250,000
Wigwag or flash signals.............................	500,000
24-hr. watchmen or gates...........................	700,000

To illustrate, if a crossing has a danger index of 50,000, the probability of serious accident is obtained as follows:

$$\text{Unprotected conditions } \frac{50{,}000}{125{,}000} = 0.4 \text{ accident per year.}$$

$$\text{Flash signals.......... } \frac{50{,}000}{500{,}000} = 0.1 \text{ accident per year.}$$

Probable yearly accident damages can be approximated from this for any assumed average injury amount, say, \$15,000 per serious injury.

The use of a danger index appears to some engineers at first sight as of little real value and a complicated matter. In reality, it is simple and has been found to agree with average accident records for a 10-year period and with the common-sense judgment of railroad and highway engineers. It has been adopted as the official standard of rating the crossings for order of elimination improvement. The factors are explained as follows:

Highway-traffic Factor.—This is obtained from the yearly traffic census modified for probable 24-hr. year-round daily volume and corrected for probable increase during the period to be considered (see Chap. I, p. 32).

Railroad-traffic Factor.—For grade-crossing danger index the following values are used, which consider speed and delay congestion:

Number passenger trains per 24 hr. full speed × 1.
Number passenger trains per 24 hr. near stops × 0.25.

Number freights full speed × 0.5 Number freights near stops × 0.25	highway traffic less than 2000 daily.
Number freights full speed × 0.75 Number freights near stops × 0.25	highway traffic 2000 to 4000 daily.
Number freights full speed × 1.0 Number freights near stops × 0.25	highway traffic over 4000 daily.

Visibility Factor.—For good visibility where driver of road vehicle can see 500′ each way along track when he gets to within 70′ of the crossing a factor of 1.0 is used.

For a view of less than 500′ from each of the four possible directions of sight 0.1 is added to the factor, giving a maximum factor of 1.4.

The effect of visibility of approaching trains is very indefinite and there is no agreement among engineers as to values to be assigned. It is certain, however, that it is not directly proportional to degree of poor sight. All drivers use more care at a blind crossing than at an open crossing unless the crossing is protected by watchmen or signals, in which case the invisibility of approaching trains has little effect on accident.

Where the crossing is unprotected it is necessary for the driver to see approaching trains with sufficient ease while he is still far enough away from the crossing to get his car under control. It is also necessary for him to get across the tracks, after the last time he looks for trains, before an approaching train reaches the crossing. The author's personal investigations of the habits of ordinarily careful drivers on improved roads (state system) indicates that they cross the usual crossing at about 20 to 25 miles per hour and that the last time they look for trains is about 60 to 80′ from the first rail. If they see no trains they confine their attention to driving. If these assumptions are correct, safe visibility must be sufficient to permit the road vehicle to travel up to and across the tracks before a train, which could not be seen the last time the driver looked, reaches the crossing.

For usual single- or double-track crossings the road vehicle must travel 100 to 200′ while the driver is not looking for trains. At different speeds covering this distance would take the following times:

Speed, miles per hour	Time of crossing	
	100′ distance, in seconds	200′ distance, in seconds
10	7	14
15	4.5	9
20	3.5	7
25	2.7	5.4
30	2.3	4.6

The distance trains travel per second at different speeds is approximate as shown in the following table.

Speed, miles per hour	Feet per second	Required visibility for a six-second interval, in feet
20	30	180
30	45	270
40	58	350
50	73	450
60	88	530

A leeway of six seconds for a sight distance 70′ from the track should be the minimum safe interval for ordinary conditions. This interval requires visibility lengths (shown in the last column of the above train speed table) according to the speed of the approaching train.

Approach Factor.—For good approach conditions use factor of 1.0. If approach conditions are worse than the minimum requirements of specifications given on page 609, 0.1 is added for each side of crossing which does not meet specifications, giving a maximum factor of 1.2. The main danger is due to difficulty in stopping cars on a downhill grade when the pavement is slippery.

For example of figuring danger index see sample report p. 606.

ELIMINATION OF GRADE CROSSINGS

Order of Importance.—Grade crossings can be eliminated by means of grade separations and relocations. Road relocations can often be made at moderate cost, but grade separations are always an expensive proposition. As funds are always limited for such work, it is important to select the crossings for first consideration which give the most reduction in danger and delay per dollar expended, that is, any large program should be coordinated for relative danger, considering the installation of signal or watchman protection, and for best business investment, considering elimination construction. This can be readily approximated by means of careful preliminary estimates of cost of the most feasible methods of elimination in conjunction with the relative danger at the present

(text continued on page 630.)

TABLE 114A.—ELIMINATION PROJECTS ARRANGED IN ORDER OF GREATEST DANGER, STEAM RAILROADS

Order greatest danger	Danger index	Index map number of crossing	County	Town	Railroad	Branch railroad	Highway number or name
Road relocation (no structures)							
1	38,000	22 & 202	Livingston	Avon	Erie	Avon-Mt. M.	623 & 1030
2[a]	21,000	65	Livingston	Groveland	D. L. & W.	M. L.	8112
3[a]	20,000	(F.A.)85 & 205	Livingston	Nunda	Erie	M. L.	Dalton-Caline
4	10,500	172	Monroe	Mendon	L. V.	M. L.	1187
Existing dangerous eliminations							
1	10,000	163	Monroe	Perinton	N. Y. C.	West Shore	60
2	9,000	43	Orleans	Ridgeway	N. Y. C.	Falls Br.	5469
3	8,000	(F.A.)88	Genesee	Stafford	L. V.	M. L.	5145
4	6,000	24	Livingston	Conesus	Erie	Corning	828
5	2,200	52	Livingston	Groveland	Penn.	Roch.	9100
6[a]	2,000	182	Livingston	York	D. L. & W.	Switch	Retsof
7	1,500	120	Monroe	Greece	N. Y. C.	Falls Br.	148
Program for signals, watchmen, etc.							
1[b]	220,000	1[b]	Monroe	....	N. Y. C.	Main line	166
2[b]	180,000	142[b]	Genesee	Bat. City	N. Y. C.	Main line	Bat. City
3[b]	180,000	(F.A.) 143[b]	Genesee	Bat. City	N. Y. C.	Main line	Bat. City
4[b]	150,000	99[b]	Monroe	....	N. Y. C.	Main line	1307
5[b]	100,000	118[b]	Monroe	....	N. Y. C.	Main line	574
6[c]	81,000	7[c]	Monroe	Henrietta	N. Y. C.	West Shore	62

7[b]	80,000	2[b]	Monroe	....	L. V.	Main line	648
8[d]	75,000	(F.A.)34[d]	Monroe	Riga	N. Y. C.	West Shore	634
9[b]	75,000	138[b]	Genesee	Le Roy Br.	N. Y. C.	Main line	Le Roy Br.
10[b]	73,000	(F.A.)4[b]	Genesee	....	B. R. & P.	Roch Div.	Ler. Village
11[e]	60,000	32[e]	Monroe	Chili	N. Y. C.	West Shore	79
12[b]	60,000	(F.A.)116[b]	Monroe	....	N. Y. C.	West Shore	5435
13[b]	43,000	11[c]	Genesee	....	N. Y. C.	West Shore	1402
14[ae]	40,000	(F.A.)6[e]	Monroe	....	N. Y. C.	Auburn Br.	776
15[e]	40,000	8[c]	Monroe	Henrietta	L. V.	Roch. Div.	62
16[e]	40,000	69[e]	Monroe	Gates	Penn.	Switch	63
17							
18[d]	38,000	17[f]	Orleans	Murray	N. Y. C.	Falls	5025
19[b]	36,000	159[b]	Monroe	....	N. Y. C.	Main line	634
20[b]	30 000	160[b]	Monroe	....	N. Y. C.	Main line	1236
21[b]	[illegible]	161[b]	Monroe	....	N. Y. C.	Main line	1390
22[b]	36,000	(F.A.)195[b]	Livingston	....	D. L. & W.	Main line	1248
23[d]	33,000	(F.A.)5[f]	Ontario	Farmington	N. Y. C.	Auburn	484
24[b]	30,000	57[b]	Ontario	....	L. V.	Main line	1415
25[b]	30,000	(F.A.)105[b]	Ontario	....	N. Y. C.	Falls & B. C.	Cana. City

[a] Means construct when road is improved.
[b] Means adequate signals exist.
[c] Unprotected.
[d] Means existing signals insufficient.
[e] Means eliminations ordered.
[f] Existing signals fairly good.

NOTE. Letters F.A. before map number means on federal-aid system.

NOTE. Condition of signals tabulated for Jan., 1923.

TABLE 114B.—ELIMINATION PROJECTS ARRANGED IN ORDER OF BEST BUSINESS INVESTMENT, STEAM RAILROADS

Order best business investment	Investment index	Index map number of crossing	County	Town	Railroad	Railroad of branch	Highway number or name
Road relocations (no structures)							
1[a]	(F.A.) $0.50	85 & 205	Livingston	Nunda	Erie	M. L.	Dalton-Hornell
2	1.60	22 & 202	Livingston	Avon	Erie	Avon-Mt.M.	623 & 1030
3[a]	1.65	65	Livingston	Groveland	D. L. & W.	M. L.	8112
4	1.90	172	Monroe	Mendon	L. V.	M. L.	1187
Existing dangerous eliminations							
1[a]	$ 5.00	182	Livingston	York	D. L. & W.	Switch	Retsof
2	5.00	88(F.A.)	Genesee	Stafford	L. V.	Main line	5145
3	6.80	43	Orleans	Ridgeway	N. Y. C.	Falls Br.	5469
4	10.00	24	Livingston	Conesus	Erie	Rochester Corn.	828
5	10.00	163	Monroe	Perinton	N. Y. C.	West Shore	60
6	33.00	120	Monroe	Greece	N. Y. C.	Falls Br.	1481
7	45.00	52	Livingston	Groveland	Penn.	Rochester Div.	9100
New eliminations, subways or overhead (on present system, 1924)							
1[b]	$0.85[c]	1[b]	Monroe	Brighton	N. Y. C.	Main line	166
2[b]	1.25[c]	99[b]	Monroe	Perinton	N. Y. C.	Main line	1337
3	1.30[c]	2	Monroe	Rush	L. V.	Main line	648
4	1.60[d]	7	Monroe	Henrietta	N. Y. C.	West Shore	62
5	1.75[e]	34(F.A.)	Monroe	Riga	N. Y. C.	West Shore	634
6	2.20[c]	4(F.A.)	Genesee	Le Roy	B. R. & P.	Roch. Div.	Le Roy

7[f]	2.50[e]	6[f](F.A.)	Monroe	Pittsford	N. Y. C.	Auburn	766
8[f]	2.25[e]	11[f]	Genesee	Oakneld	N. Y. C.	West Shore	1402
9[b]	2.65[c]	138[b]	Genesee	Bergen	N. Y. C.	Main line	Le Roy-Bkport
10	2.75[d]	8	Monroe	Henrietta	L. V.	Roch. Div.	62
11	2.85[d]	32	Monroe	Chili	N. Y. C.	West Shore	79
12[b]	3.25[d]	69[b]	Monroe	Gates	Penn.	Switch	63
13	3.50[c]	60	Genesee	Bethany	D. L. & W.	M. L.	1268
14	3.60[g]	17	Orleans	Murray	N. Y. C.	Falls Br.	5025
15	3.75[e]	70(F.A.)	Livingston	Geneseo	Erie	Avon-Mt. M.	1247
16	3.90[c]	160	Monroe	Chili	N. Y. C.	M. L.	1236
17	4.15[c]	3	Monroe	Mendon	L. V.	M. L.	493
18	4.20[e]	161	Monroe	Gates	N. Y. C.	M. L.	1390
19[b]	4.20[c]	195[b](F.A.)	Livingston	Leiscester	D. L. & W.	M. L.	1248
20	4.25[g]	5 & 516(F.A.)	Ontario	Farmington	N. Y. C.	Auburn & Elect.	484
21[h]	4.25[h]	(F.A.)12[h]	Genesee	Alexander	D. L. & W.	M. L.	8177
22	4.60[d]	38(F.A.)	Livingston	Caledonia	Penn.	Roch. Div.	5273
23	4.65[e]	58	Genesee	Elba	N. Y. C.	West Shore	8041
24	4.70[d]	20(F.A.)	Wyoming	Gainesville	Erie	Buf.-Corn.	5400A
25	4.80[c]	72	Livingston	Mt. Morris	D. L. & W.	M. L.	622 & 855

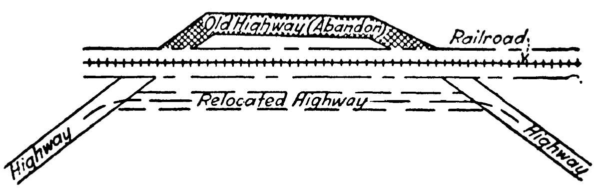
[a] Means construct when road is improved.
[b] Means located in villages or bad physical conditions which will normally delay construction.
[c] Means adequate signals exist.
[d] Unprotected.
[e] Means existing signals insufficient
[f] Means ordered by Public Service Commission.
[g] Existing signals fairly good protection.
[h] Means constructed.
Note. Letters F.A. before map number mean on federal-aid system.

crossings estimated in some manner approximately similar to the danger-index method given on page 623. A relative best business-investment index can then be established for each crossing by dividing the cost of construction by the danger index and the crossings arranged in order of relative value (see tabulation 114*B*).

To illustrate again from western New York where this method is in official use on the state road system program the following is quoted from the official report on the Proposed Elimination Program:

"The general method of determining the relative value of elimination projects is based on dividing the cost of the proposed elimination for each crossing, which ranges from $50,000 to $250,000, by the danger-index number for that crossing, which ranges from 200,000 to 500. While this, of course, is not rigidly applicable, it divides the crossings into rough general classifications of relative effective expenditure, as the index numbers ranged from 40 cts. to $160 for new eliminations and from $5 to $40 for the improvement of dangerous existing eliminations. All crossings in the division were then tabulated in the order of their best business-investment order, dividing the tabulation into two parts, one for new eliminations and one for the improvement of dangerous existing eliminations. The crossings for each railroad were similarly tabulated. The crossings in each county were similarly tabulated. In this way expenditures can be apportioned to counties and to the different railroads without undue favoritism, and at the same time the most important crossings selected for first consideration. The value of such data has been amply proved in this division during 1924 in connection with recommendations for action in six petitions. Based on this program we advised quick action on three projects near the top of the list at an estimated total cost of $225,000, which are well worth the expenditure, and advised indefinite delay with signal protection on three projects well down on the list which are comparatively unimportant and would have cost $300,000."

Consolidation of Crossings and Eliminations by Relocation. In many cases highways which are traveling in the same direction

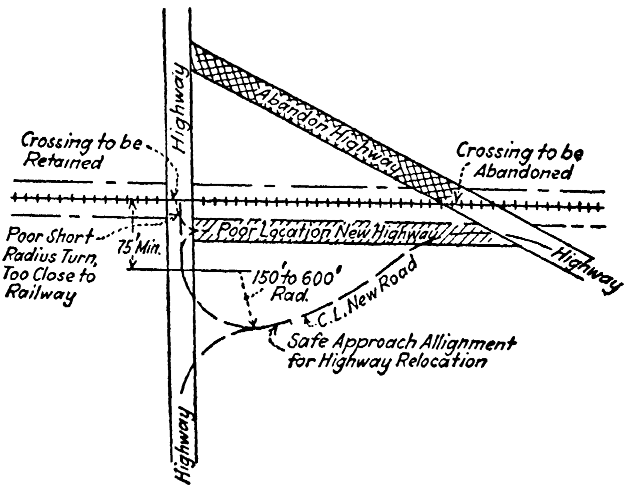


SKETCH A.—Relocation elimination.

as a railroad cross the railroad needlessly as shown in sketch *A*. For such cases it is obviously desirable to relocate to avoid crossing provided it is physically possible and provided the new highway is properly designed for easy alignment, grades, widths, etc.

In many cases it is possible to consolidate crossings as shown in sketch *B*. This method is not necessarily desirable, nor does it necessarily reduce danger unless the number of times a vehicle crosses the railroad is reduced or unless the crossing which is retained is an easier, better protected, and safer crossing than the one abandoned. This requires careful engineering location and excellent design of the new proposed approaches of the highway relocation. That is, the use of one crossing for the same amount of traffic as previously handled by two crossings is just as dangerous as the original two crossings unless approach and protection conditions are improved. Accident records show that accidents are approximately directly proportional to the volume of highway traffic for any given condition of sight distance, approach grades, visibility, and number and speed of trains.

Sketch *B* shows examples of both poor and good relocation approach layouts. The poor layout is often advocated by the railroads as they require less right of way. Poor location of this



SKETCH B.—Relocation consolidation of crossings.

character are extremely dangerous as drivers cannot see trains approaching from the rear, and the short abrupt turn at the crossing gives them little opportunity to look up the track before they are on the crossing.

The long radius balloon-curve approach with a minimum tangent of 75′ from the track makes the approach safe. The radius of the approach curve should be varied according to the relative importance as required (see alignment specifications, p. 120).

See page 775 for official report on a relocation proposition.

SUMMARY OF GENERAL PRINCIPLES OF ELIMINATION OF CROSSINGS BY MEANS OF SIDE-LINE CONSOLIDATIONS

1. The crossing retained must be safer than the crossing abandoned.
2. The road design of the new connecting road must be up to reasonable engineering standards and must be particularly good in regard to approach alignment and grades at the retained crossing, permitting good car control and ample time for drivers to sense the approach of trains or to see automatic signals.

3. If there is a large difference in highway traffic volume on the two roads to be consolidated, the crossing on the heaviest traffic highway should be the one retained and improved.

4. If the traffic on both roads is practically the same and is light volume and if the crossings are of equal safety, the retained crossing should be selected to result in the least increase in distance of travel for highway traffic.

5. If the traffic is practically the same for both roads and is heavy traffic, it is often better to protect each crossing by automatic signals or by eliminations rather than to attempt to consolidate them, on account of congestion, added distance, and inconvenience to traffic. That is, it is not permissible to consolidate crossing solely on the score of reducing the number of crossings when such action is a distinct detriment to the efficiency of the highway layout.

Subway and Overhead Elimination Specifications.—As structural eliminations are expensive and comparatively permanent, they should be carefully designed to provide adequate safety and comfort for both highway and railroad traffic. Provision should also be made for additional railroad trackage and a highway roadway width which will handle the expected growth in volume of travel during the life of the structure.

In the past there have been too many cases where such eliminations were designed solely from the railroad standpoint of getting rid of collisions at grade crossings with a consequent reduction in damage claims and without any serious consideration of safe highway conditions at the new elimination. That is, steep highway grades, crooked alignment, and narrow bridges at poorly designed eliminations have resulted in as many serious injuries to highway users as occurred at the old grade crossings due to collisions.

To illustrate the uselessness of poorly designed structural eliminations in reducing accidents the following accident statistics for 1926 on the state highway system in western New York (Division 4) are cited.

215 grade crossings—15 serious injuries or 0.07 injury per crossing average
10 poor dangerous eliminations, sharp curvature and narrow bridges, 4 serious injuries or 0.4 injury per elimination
16 fairly good eliminations, 3 serious injuries or 0.2 injury per elimination
29 good eliminations—A few minor accidents but no serious injuries.

As a general rule, subways (highway under railroads) are safer than overhead crossings. The main reasons for demand for eliminations in this territory are to give free and safe access to new real estate development territory and to prevent delay and congestion on heavily traveled roads.

Provisions for future traffic growth is a vital part of the design. Too much liberality results in running the initial cost up beyond reason, and too little provision results in congestion and danger in a short period. As eliminations *in rural districts* are really luxuries instead of necessities they may better be dispensed with entirely unless they are made safe for a reasonable term of years. It is poor policy to run the initial cost up needlessly by too liberal design for future requirements for any portion of the structure which can be gradually widened to take care of future traffic growth without necessitating the complete rebuilding of the entire structure. That is, approach fills can always be widened at any time to get additional traffic lanes. Stringer type overhead bridges can be widened at any time without rebuilding the entire bridge. Additional trackage can be provided through a trestle approach span at any time by the construction of special walls which in no way disturb the old structure. The author has been in the habit of allowing for

20 years' traffic growth for portions of the design which can be expanded without loss and allowing for 50 years' growth for structures which will have to be rebuilt entire to widen (see p. 32 for discussion of future traffic).

The following specifications indicate minimum requirements necessary for reasonably safe and convenient elimination designs.

SPECIFICATIONS FOR GRADE-CROSSING STRUCTURAL ELIMINATIONS

NOTE.—In the following specifications "subway" means where the highway passes under the railroad and "overhead" means where the highway passes over the railroad. The daily traffic volumes given refer to 12 hour daylight counts taken on Friday and Saturday in August.

General Specifications

As a general rule the elimination layout shall conform to normal highway ocation and alignment. The introduction of curved highway approach alignment solely for the purpose of reducing span and cost of bridge structures is rarely justified.

As a general rule, subways are to be preferred to overhead crossings on the score of highway safety, appearance in villages, and urban property values unless the cost is materially greater than an overhead elimination. Subway eliminations, however, must not have piers or column obstructions within the pavement area.

Fill approaches for overhead crossings are preferred to trestle approaches unless cost is materially increased due to property damage. This does not apply to single-approach trestle spans to reduce abutment costs.

Bridge Specifications and Clearances

Subway Clearances (Railroad Bridges):	
Vertical clearance, crown of road to bottom of railroad bridge.	
Main roads	14′
Secondary roads	13.5′
Width of roadway at right angles to highway center line:	
Main roads (over 9,000 vehicles daily),	42′ minimum
Main roads (6,000 to 9,000 vehicles daily),	32′ minimum
Main roads (up to 6,000 vehicles daily),	26′ minimum
Unimportant roads (up to 300 vehicles daily),	20′ minimum
Sidewalks where needed,	5′ minimum
Overhead Elimination Clearances (Highway Bridges):	
Vertical clearance, top of rail to bottom of highway bridge:	
Steam railroads:	
Normal	22.0′
Minimum	21.0′
Electric trolleys:	
Normal	16.0′
Minimum	14.0′
Highway bridge portal clearances:	
Main roads	14.0′
Secondary roads	13.5′
Side railroad clearances, outside rail to face of highway bridge abutment or pedestals:	
Steam railroads:	
Straight main track, minimum	8′
Usual practice	10′
Straight siding track, minimum	6′
Curved main track, minimum	10′
Electric trolley lines:	
Straight track, minimum	4′
Curved track, minimum	6′
Railroad bridge widths:	
Center to center of girders single track on tangents.	15′ 06″ to 17′ 06″ ±
Center to center of girders double track on tangents.	30′ 06″ ±

Increase for curvature and length of span.

Highway bridge roadway widths, minimum:

Main roads (over 9,000 vehicles daily)	40′
Main roads (6,000 to 9,000 vehicles daily)	30′ minimum
Main roads (up to 6,000 vehicles daily)	24′ minimum
Unimportant roads (up to 300 vehicles daily)	20′ minimum
Sidewalks as needed in villages and all roads carrying over 2,000 vehicles daily, minimum	5′

Depth of Bridge Floors (Approximate Only for Preliminary Layouts):

Railroad plate girders, bottom rail to bottom of girders (through girder):

Steam railroads	3.5–4.0′
Electric railroads	3.0–3.5′

Deck girders 1/12 of span plus 18″:

Overhead highway bridges	3.0–4.0′

NOTE.—Overhead highway bridge floor system encased in concrete to protect from engine gases for main span over track. Use stringer type of bridge up to 55′ clear span to permit easy future widening.

All railroad bridges to have solid floors to prevent falling of objects onto road.

Bridge Design Loadings:

Highway bridges:	
Main roads	H-20
Secondary roads	H-15
Railroad bridges:	
Steam (main lines)	E-70 or E-60
Steam (branch lines)	E-60 or E-50
Electric interurban	E-40 to E-30

Highway Approach Specifications

Minimum Sight Distance:

Main roads	350′
Secondary roads	250′

Alignment:

Main roads (straight if possible):	
Minimum center-line radius:	
On grades less than 5%	570′
On grades 5% or greater	800′
Side road intersections, minimum:	
Radius of curvature	50–350′
(See p. 121 for discussion of intersections)	
Driveway connections, radius	25–35′
(See p. 121 for discussion of drive alignment)	

Maximum Grades:

Subways:	
Main roads, good alignment	6.0%
Secondary roads, good alignment	7.0%
Overheads:	
Main roads, good alignment	5.0%
Secondary roads, good alignment	6.0%
Electric railroad maximum grade	4.0%

NOTE.—Reduce grades for alignment sharper than stipulated above. A combination of a 6% grade on a 7 deg. curve (800′ radius) is the maximum permissible combination which serves satisfactorily. If sharper curvature is needed reduce grade. (See p. 118 for discussion.)

Minimum Length of Vertical Curves:

100′ for each 4% difference in gradients at bottom of hills

125′ for each 4% at top of hills

Carry road grade vertical curve across overhead bridges and make floor surface coincide with this vertical curve

Roadway Widths (Minimum):

Pavements:	
Main roads, over 9,000 vehicles daily	40′
Main roads, 6,000 to 9,000 vehicles daily	27′
Main roads, 4,000 to 6,000 vehicles daily	20′
Main roads, up to 5,000 vehicles daily	18′
Unimportant roads	16′

Pavement under railroad bridges full width abutment to abutment, for length of runoff to normal width see page 133.

Grading:
- Ditch to ditch in cut:
  - Two-lane traffic .... 26–32′
  - Three-lane traffic .... 36–42′
- Top edge to edge of fills:
  - Two-lane traffic, up to 6,000 vehicles .... 32′
  - Three-lane traffic, 6,000 to 9,000 vehicles .... 42′
  - Unimportant roads .... 24′

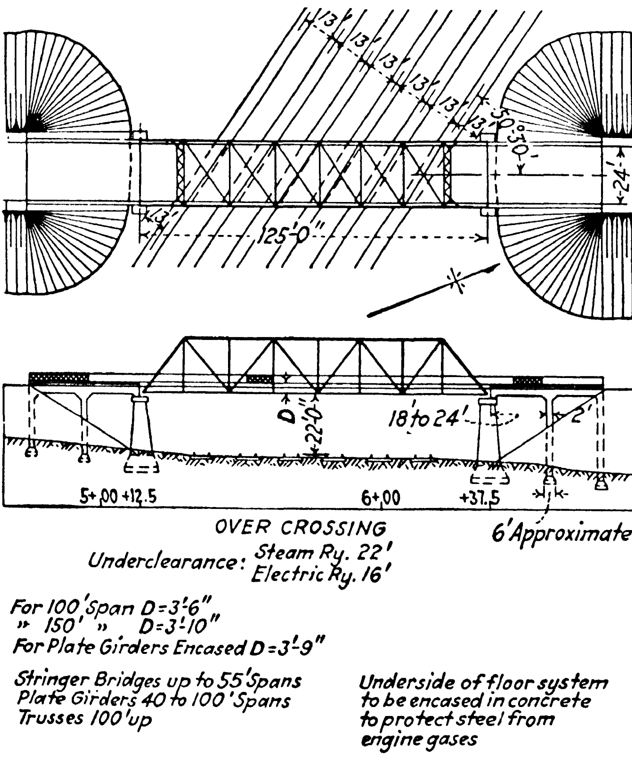


FIG. 203.—Typical overhead highway bridge clearances.

Sidewalks: Complete segregation from roadway by pipe rail and high raised curb (at least 12″ in subways) and by means of a separate lane fenced off by substantial rail on overhead fills.

Pavement Types: Main roads over 2,000 vehicles daily. Use rigid type of pavement in subways. Use temporary flexible type of pavement (gravel or macadam) on deep approach fills, later replaced with a rigid type after fill has settled.

(text continued on page 641.)

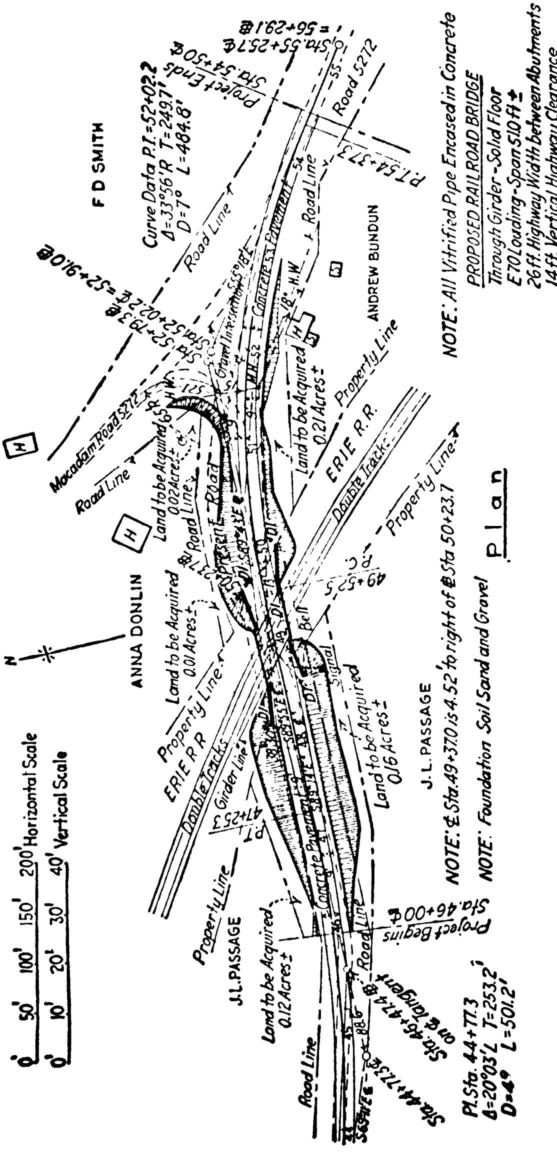


FIG. 204.—Typical subway layout Highway under railroad Rural road conditions.

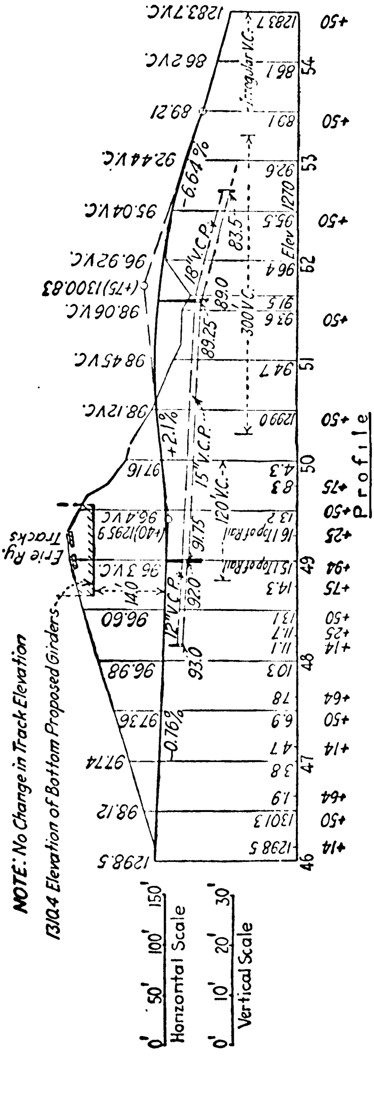


FIG. 204.—(*Continued.*)

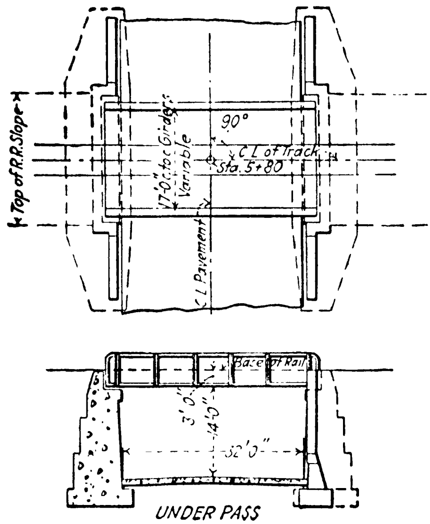


FIG. 204A.—N. Y. State standard underpass clearances rural state highways (1926).

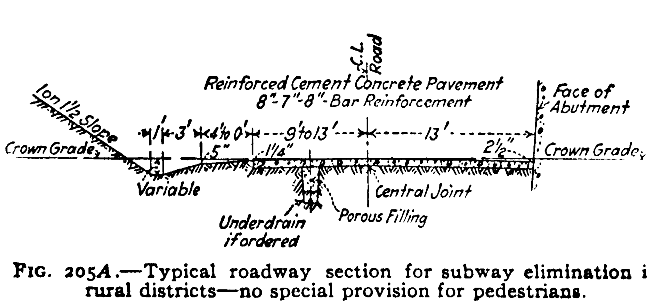


FIG. 205A.—Typical roadway section for subway elimination in rural districts—no special provision for pedestrians.

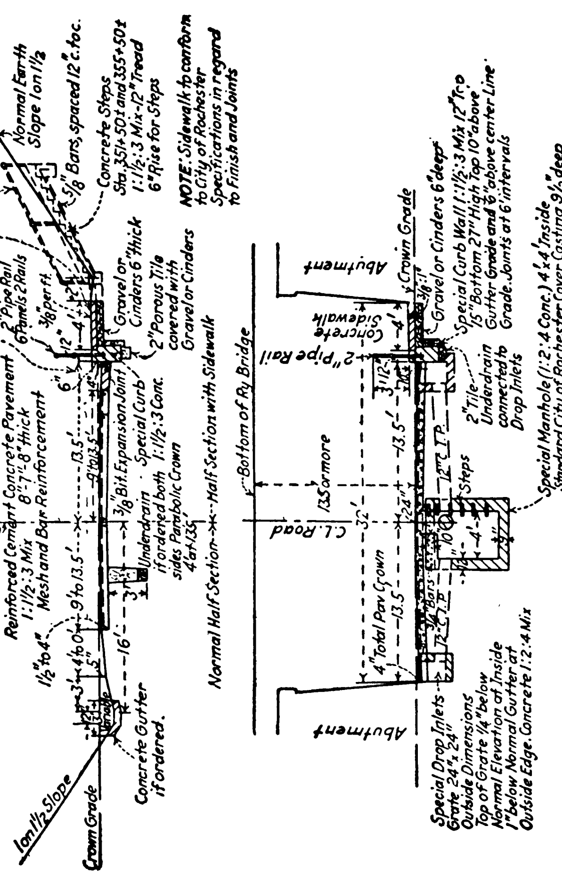


FIG. 205B.—Typical subway road sections pedestrian traffic rural districts.

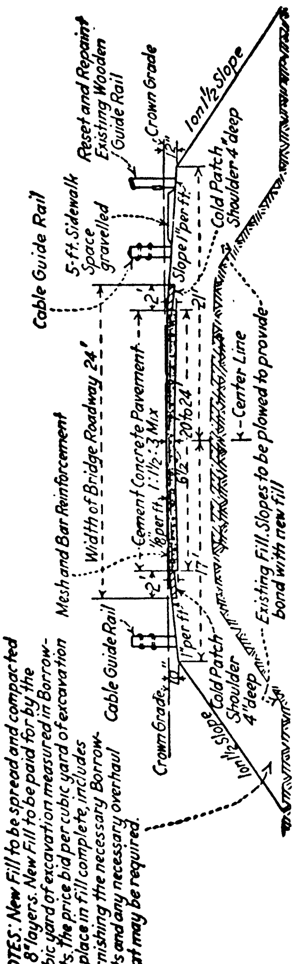


FIG. 206.—Typical roadway section for overhead elimination on main road with provision for pedestrians.

SPECIFICATIONS.—*(Continued.)*

Secondary roads, up to 2,000 vehicles daily, macadam or gravel.

NOTE.—For depths of pavements on different soils and for different traffic see Table 74, page 391 and Table 86, page 426.

Drainage: In subways pick up water on both sides of bridge to keep low spot dry. Never run water through on the surface and collect on one side unless road grade is on a continuous down grade for the entire elimination approaches.

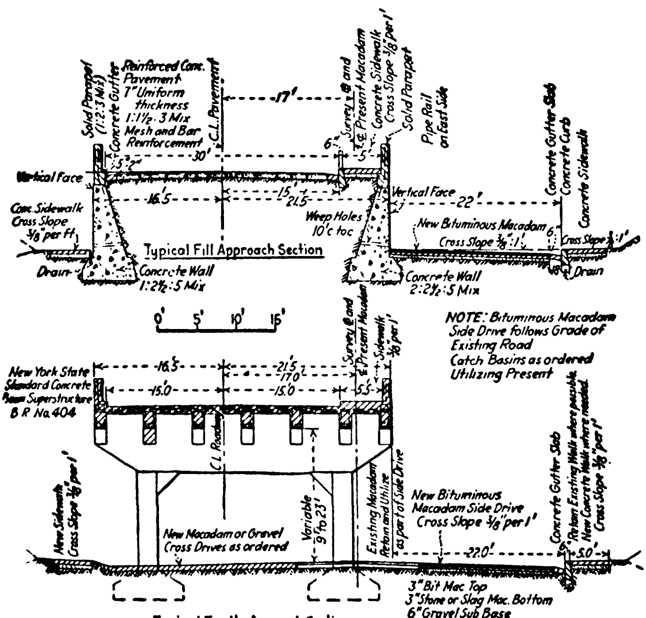


FIG. 206A.—Typical rural roadway approach section for overhead highway eliminations. (Village conditions.)

Miscellaneous Track and Clearance Data.—For side and vertical clearances see Specifications (p. 633).

The following typical roadway sections and track gage and center-line spacing are useful in connection with layouts.

TABLE 115.—SPACING OF TRACKS C TO C CENTER AROUND CURVES TO ALLOW FOR OVERHANG OF CARS AND OBTAIN EQUIVALENT TO THAT ON STRAIGHT TRACK[1]

Centers on tangent track		Curvature in degrees															
		1		2		3		4		5		6		7		8	
Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches
12	00	12	1	12	3	12	5	12	6	12	8	12	9	12	10	13	0
12	6	12	7	12	9	12	11	13	0	13	2	13	3	13	4	13	6
13	00	13	1	13	3	13	5	13	6	13	8	13	9	13	11	14	0

Centers on tangent track		Curvature in degrees													
		9		10		12		14		16		18		20	
Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches
12	00	13	2	13	4	13	6	13	9	14	1	14	4	14	6
12	6	13	8	13	10	14	0	14	3	14	7	14	10	15	0
13	00	14	2	14	4	14	6	14	9	15	1	15	4	15	6

[1] *Weiss*, Practical Railway Maintenance, McGraw Hill Book Company, Inc.

Preliminary Investigations.—Preliminary investigations cover trial estimates for all feasible schemes of elimination to pick the best and most economic solution. They must consider the possibilities of road relocations, subways or overhead methods, different combinations of alignment and grade property damage, drainage, pavement types, etc. These investigations should be made by experienced engineers, as they set the general plan and control the economics of design. A sample report follows and is supplemented by various diagrams for quick estimating in comparing different possible layouts.

MEMORANDUM FOR ———, PROPOSED GRADE-CROSSING ELIMINATION, CROSSING 34, STA. 2 + 00 + CHURCHVILLE VILLAGE, C. H. 634, NEW YORK CENTRAL RAILROAD (WEST SHORE BRANCH) MONROE COUNTY

"**Recomendation.**—I recommend that this crossing be eliminated in 1926 by means of an overhead highway crossing.

"**Cost.**—The cost of this elimination, including property damage, is estimated at $133,000.

"The cost of the highway approaches and temporary side road is estimated at $63,000; the bridges and trestle at $52,000.

"The transmission line work at $5000 and property damage at $13,000.

"The preliminary program estimate of February, 1924, was $120,000, including property damage. The preliminary program estimate of March, 1925, was $130,000.

"**Status of Crossing.**—This crossing is first in order of importance in Division 4 for new elimination projects and should be constructed in 1926, regardless of the success or failure of the pending bond issue. The con-

(text continued on page 644.)

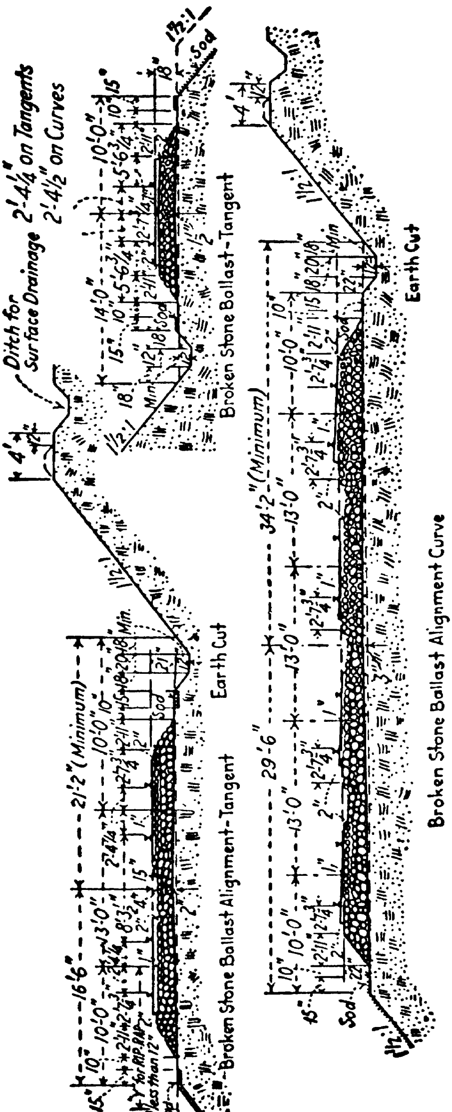


FIG. 207.—Typical railroad roadbed sections.

struction of this elimination cleans up all crossings on the main Rochester-Buffalo road between Rochester and Batavia.

"This crossing has a danger index of 75,000 and an investment index of $1.75.

"The immediate elimination of this crossing has been formally requested by the Rochester Chamber of Commerce, the Rochester Auto Club, the county of Monroe, the town of Riga, the Batavia Auto Club, and the county of Genesee.

"Physical Conditions at Crossing.

Number of tracks	2
Balast	stone
Weight of rail	110
Alignment of railroad	straight
Approximate track grade	level
Number of passenger trains	4
Speed of passenger trains	30 m.p.h.
Number of freight trains	40±
Speed of freight trains	25 m.p.h.
Present crossing protection	Watchman 8:30 a.m. to 5:30 p.m.
Angle center line and center-line road	36°±
Highway approach grades west	−3.3% east + 0.8%
Present highway pavement	Topeka on concrete 16′ wide

"Highway Traffic (12-hr. Counts in August).

	Year		Number of Vehicles
	1920		450
	1923		405
	1924		1677
Est.	1926	Est.	2000 to 2500 on completion of route to Batavia

"Visibility of Approaching Trains.

50′ east of tracks, north	5000′, south	3000′
100′ east of tracks, north	600′, south	600′
50′ west of tracks, north	2000′, south	2000′
100′ west of tracks, north	1000′, south	200′

"Accident Record.—Two minor accidents in 10 years.

"Photographs.—Photographs attached at end of report.

Design Report

"Type of Elimination.—It is possible to construct either an overhead or a subway elimination.

"The overhead type is recommended as the most economical solution.

Estimated total cost overhead, including property damage....... $133,000
Estimated total cost subway, including property damage......... 165,000

"While a subway is feasible, the drainage is difficult, requiring an expensive sewer about 2800′ long estimated to cost approximately $23,000

"The abutting property owners prefer a subway, but all this property is residential, of low valuation. Considering the fact that residential property of this kind next to a railroad has little value and that the total cost of the overhead is materially less than for a subway, it seems reasonable to adopt the overhead type of elimination for this project.

"Fill approaches on the west are obviously the proper designs. Fill approach with a short retaining wall on the east is adopted, as it saves from $25,000 to $30,000 over a trestle-approach design, allowing for a slight increase in property damage for three owners. This large saving apparently warrants the design, as the extra property damage is small. The fill approach is also more permanent than the trestle design.

"Location and Alignment. *Main Road.*—The existing road location and alignment is retained, as this is obviously the best solution, considering the village property and the importance of the highway (main Rochester-Buffalo road).

"The skew angle of crossing could be slightly reduced by reverse approach alignment, but the small saving in cost (approximately $13,000) would not warrant the adoption of the crooked approach alignment with more danger and poorer appearance.

"*Corporation Line Approach Road.*—This road has a very small amount of travel. We have relocated 500′ of this highway to increase the safety of travel at the junction of this road with the main road.

"This relocation costs about $6000 less than the retention of the existing location with a high approach fill and at the same time increases the safety of the layout.

"**Grades.**—Six per cent maximum approach grades are recommended on account of reduced property damage, 5 % is the prevailing maximum on this route, but 6 % will not materially increase danger, considering that the approach alignment is excellent.

"**Sight Distance.**—Vertical curves designed to give a 350′ minimum sight distance on the main road.

"This is the minimum desirable distance for a road of this importance.

"Minimum sight distance at intersection of main road with corporation line approach road, 350′.

"**Drainage.**—Simple—no comment necessary.

"**Roadway Section.**—Thirty-five feet top width of fill, including side path for pedestrians on north side. Width of fill can be increased at any time to handle future traffic growth.

"Provision for pedestrians adds about $8000 to the cost of this elimination, but is most certainly desirable, considering future growth of Churchville Village and the heavy volume of highway traffic.

"**Pavement.**—Reinforced cement concrete 20′ wide flared to 24′ at bridge is used, as this is the main Rochester-Buffalo road.

"Sixteen feet bituminous macadam is used on the corporation line approach road. The present road is a 10′ water-bound macadam.

"**Temporary Side Road.**—A temporary side road with a 24-hr. watchman at the crossing will be necessary, as there is no chance for a good normal road detour

"As the volume of traffic is heavy, this side road must be a good job; we have figured on 16′ width of gravel 9″ deep with a grading width of 22′ safe to drive on.

"The amount for the watchman is figured as 16 hr. per day over and above the present protection of 8 hr.

"**Transmission Line.**—The transmission line of the Niagara, Lockport & Ontario Power Co. will have to be raised. The estimated cost of this work is based on an estimate furnished by the Buffalo office of this company ($5000).

"**Bridge.**—Solid floor-plate girder main span with reinforced-concrete trestle approaches to eliminate solid abutments, pedestal and column supports; 24′ roadway + 5′ sidewalk; H-20 loading; 22′ vertical railway clearance; 10′ side clearance (outer rail to pedestal). Making allowance for 40 years growth in traffic volume would probably justify a 30 ft. Roadway.

"The use of this type of bridge reduces the cost below that required for the solid abutment type about $12,000, including the reduction in fill and pavement items.

"The road profile grade line is based on a 3.5′ depth of floor system for the main span bridge, which is the usual depth used by the New York Central bridge designers.

"A reduction in floor depth to 3.0′ would reduce the cost of the fill approaches approximately $1500. We recommend a 3.0′ depth of floor system as being an economical design, considering the total cost of bridge and approaches. (See page 206 for discussion of floor depths.)

"**Detail Estimate.**—A detail estimate of cost is attached.

"**General Procedure.**—I recommend a physical division of the work, as we can get the approach work done considerably cheaper than the New York Central Railroad, whose bridge contractors are not familiar with or well equipped for road work.

New York Central Ry. Bridge and watchman at crossing.
State of New York. Approaches including temporary side road.
Niagara, Lockport & Ontario Power Co. Transmission Line.

(*Signed*)
Grade Crossing Engineer."

PRELIMINARY REPORT ON THE PROPOSED PITTSFORD-PALMYRA GRADE-CROSSING ELIMINATION (AUBURN BRANCH NEW YORK CENTRAL RY.) STAS. 0 TO 12 OF ROAD 766, ILLUSTRATING ECONOMIC COMPARISON OF DIFFERENT GRADE LAYOUTS

"**Summarized Recommendations.**—We recommend a highway under-crossing at an estimated cost of between $85,000 to $100,000. This estimate is based on raising the railroad tracks 1½ to 3′ and using a solid-floor single track through girder railroad bridge, E-60 loading, a clearance of 14′ from the bottom of the bridge to crown of road, a 26′ right-angle clearance between bridge abutments, a 5% maximum highway approach grade on straight alignment, necessary cleaning out and deepening of creek channel with enlarged and rebuilt culverts to provide proper drainage, and a reinforced-cement-concrete pavement 7¾″ average thickness, 18′ normal width except under the bridge, where the width is 26′ to give an impervious surface between abutments.

"The decision in regard to the amount of track raise is based on drainage difficulty and the comparative total cost of this project as affected by different track raises. The following tabulation of comparative costs indicates that a 1½ to 2′ track raise seems to be a reasonable solution and we have therefore made the rough layout and preliminary design on this basis. This plan can be easily modified to conform with the ruling of the Public Service Commission, but we do not advise lowering the road grade below the elevations shown on the preliminary plan dated June 21, 1923, on account of general drainage conditions.

"On account of the heavy railroad grade to the south, it is possible that the railroad may desire to make a material raise in track grade at this crossing. However, there seems little justification for state cooperation in the cost of track raise of over 2 to 3′.

"We have recommended straight highway alignment on the score of safety. The cost, including loss of money in motor operation due to extra distance, could probably be reduced about $35,000 by the use of three reverse 10° curves, but this would add as much danger due to poor highway alignment as now exists on account of the grade crossing. An elimination based on poor highway alignment is a waste of money.

"The following detail report gives additional data in regard to the necessity for this elimination and discusses the minor features of design.

Submitted,

Assistant Engineer."

For Report on Relocation Eliminations see page 775.

Quick-estimating Data.—The following graphs and tabulations are useful for preliminary estimates in connection with the selection of the most suitable general layouts and in estimating the cost of large programs. All the data given have been used for these purposes by the author in western New York and have been found to agree with final contract plans within from 2 to 10%.

(See page 648.)

REPORT ON ELIMINATION RAILROAD 766. APPROXIMATE TOTAL COST ESTIMATES
(As affected by track raises)

Items	Estimated costs					
	No raise	1′ track raise	2′ track raise	3′ track raise	4′ track raise	5′ track raise
Steel bridge superstructure, E-60	$27,000	$27,000	$27,000	$27,000	$27,000	$27,000
Bridge abutments	24,000	24,000	24,000	24,000	24,000	24,000
Temporary track protection	9,000	9,000	9,000	9,000	9,000	9,000
Raising track	......	1,300	3,600	6,200	9,100	12,000
Drainage	9,200	8,500	7,700	7,400	7,000	6,800
Highway and bridge excavation	16,000	14,500	13,000	11,500	10,000	8,500
Highway pavement (18′)	9,000	9,000	9,000	9,000	9,000	9,000
Totals	$94,200	$93,300	$93,300	$94,100	$95,100	$96,300

Index of Quick-estimating Data

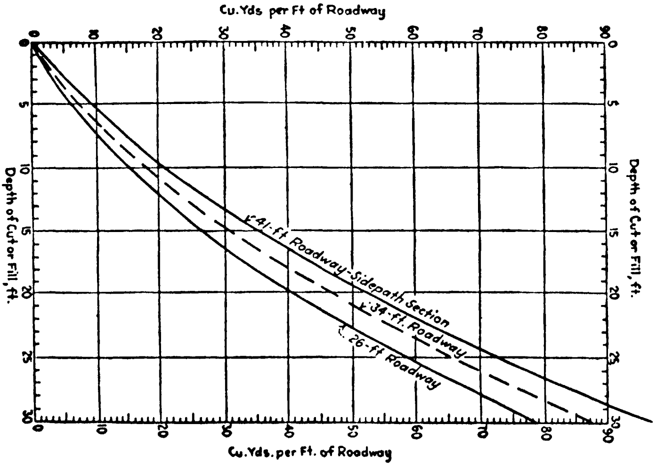
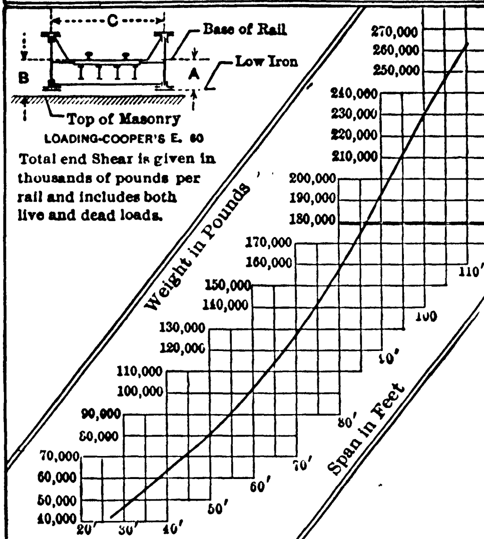


FIG. 208.—Graph for estimating road approach earthwork grade crossing eliminations.

THROUGH PLATE GIRDER SPANS, STRINGER FLOOR.
WEIGHTS, DIMENSIONS, ETC.



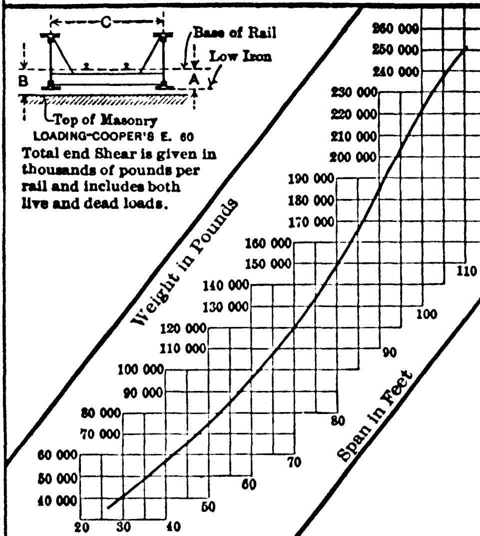
Span	Total End Sh.	A	B	C
30 Ft.	108.7	3′3″	3′3″	15′6″
35 "	121 0	3′3½″	3′3½″	16′6″
40 "	133.2	"	"	17′6″
45 "	144.7	"	"	"
50 "	156.0	"	"	"
55 "	167.5	3′4″	3′11″	"
60 "	176 3	"	"	"
65 "	191 1	3′4½″	"	"
70 "	204.7	"	"	"
75 "	220.4	"	5′0″	"
80 "	232.8	"	"	"
85 "	248.6	"	"	"
90 "	262.3	3′5″	"	"
95 "	277 4	"	"	"
100 "	291 2	3′6″	"	"
110 "	319 6	"	"	"

Erector's Note.
W = Total weight of one single track span with two light girders.

Span	Weight of one light girder.	Weight of one heavy girder.	Weight of one floor system.
30′ to 50′	.24 W	.42 W	.52 W
55′ to 80′	.25 W	.46 W	.50 W
85′ to 110′	.28 W	.51 W	.44 W

FIG. 209*A*.—Open floor bridge.

THROUGH PLATE GIRDER SPANS, I-BEAM FLOOR
WEIGHTS, DIMENSIONS, ETC.



Span	Total End Sh.	A	B	C
30 ft.	106.2	2-2"	2-2"	15-0"
35 "	117.1	"	"	16-0
40 "	128.7	"	"	17-0
45 "	141.1	2-2½	2-2½	17-6
50 "	152.2	"	"	"
55 "	162.1	2-3	2-9	"
60 "	173.2	"	"	"
65 "	186.2	"	"	"
70 "	199.3	2-3½	3-9	"
75 "	212.5	"	3-10½	"
80 "	226.4	"	"	"
85 "	240.5	"	"	"
90 "	257.4	"	"	"
95 "	268.7	"	"	"
100 "	284.0	2-4	"	"
110 "	306.0	2-4½	"	"

Erector's Note:
W = Total weight of one single track span with two light girders.

Span	Weight of one light girder	Weight of one heavy girder	Weight of one floor system
30' to 50'	.22 W	.39 W	.56 W
55' " 80'	.27 W	.48 W	.47 W
85' " 110'	.31 W	.57 W	.38 W

FIG. 209*B*.—Open floor bridge.

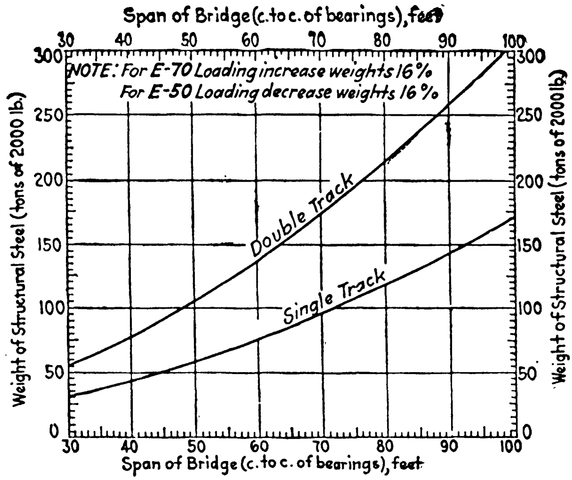


FIG. 210.—Approximate weight structural steel through girder—solid floor Ry. bridges Coopers E-60 loading.

Note: For E-70 loading increase weights 16 %
For E-50 loading decrease weights 16 %.

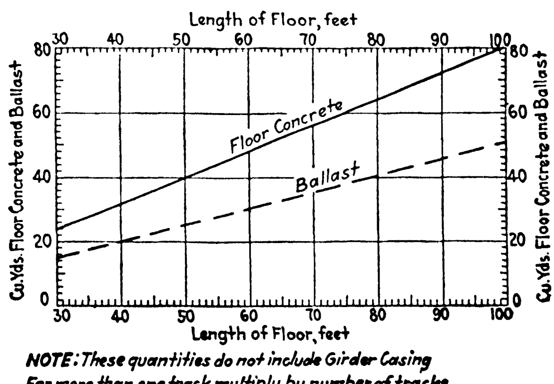


FIG. 211.—Approx. amount of floor concrete and ballast single track railroad girder bridges E-60 loading.

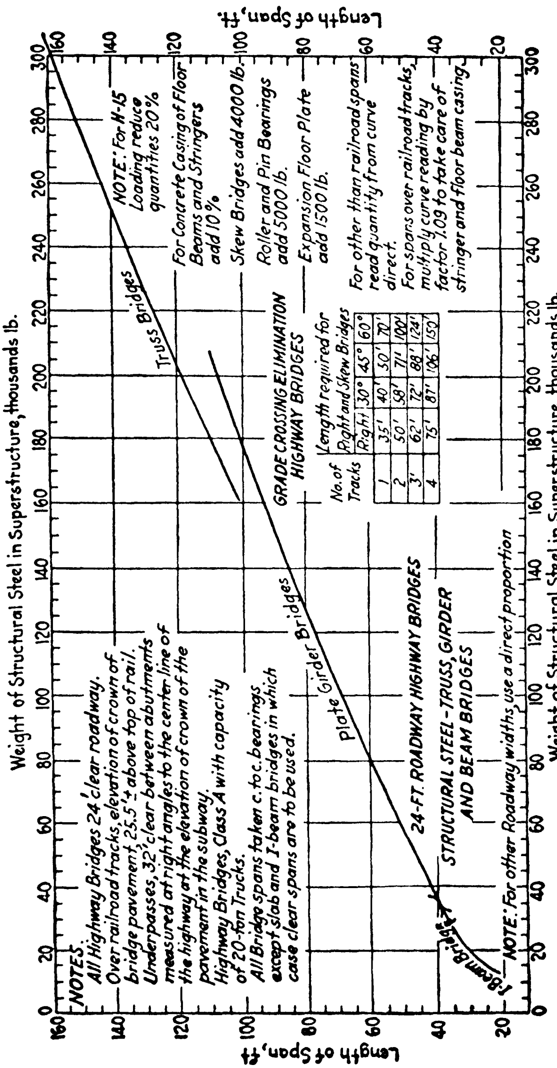
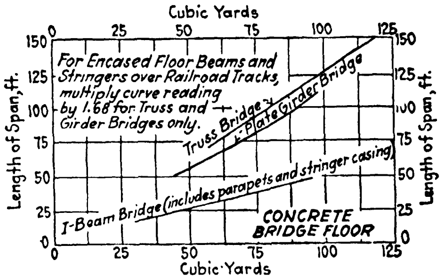


FIG. 212.—Weights of structural steel in H-20 solid floor highway bridges. (New York Standards, 1926.) (24 ft. width of roadway.)



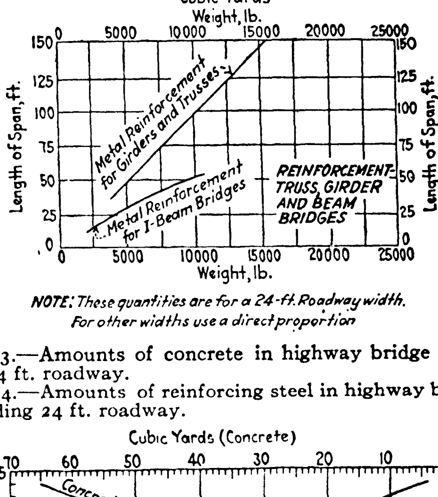


FIG. 213.—Amounts of concrete in highway bridge floors H-20 loading 24 ft. roadway.
FIG. 214.—Amounts of reinforcing steel in highway bridge floors H-20 loading 24 ft. roadway.

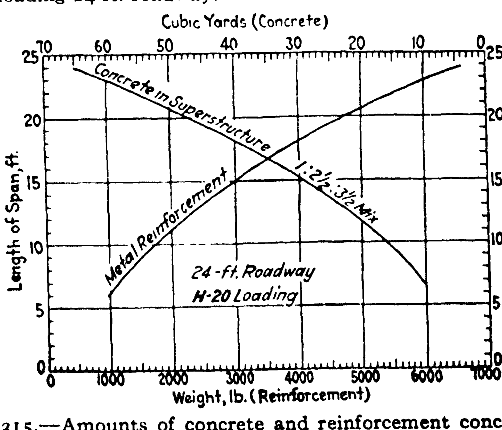


FIG. 215.—Amounts of concrete and reinforcement concrete slab superstructures. Highway bridges H-20 loading 24 ft. roadway.

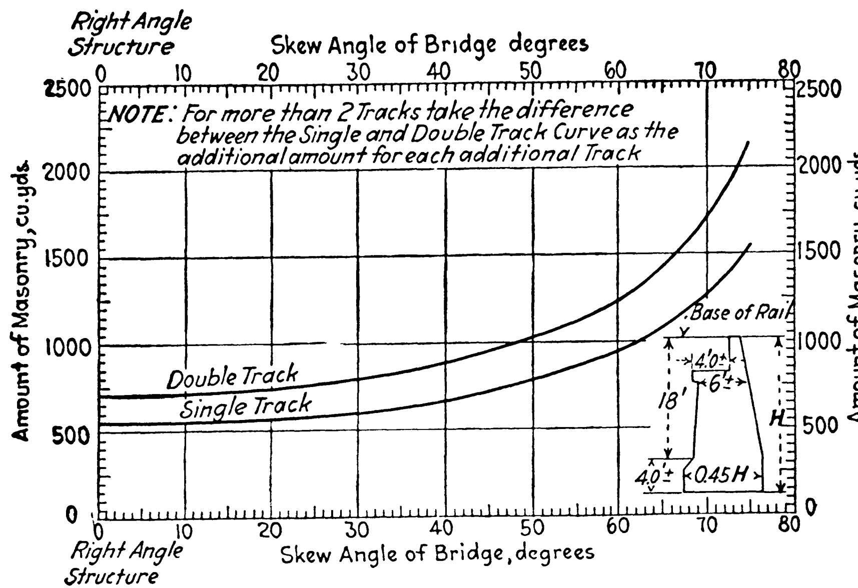


FIG. 216.—Approx. quantities for 2 abutments and 4 wings railroad bridges for usual highway underclearance of 14.0′ bottom of bridge to crown of road, which is equivalent to 18.0′ from base of rail to crown of roadway pavement.

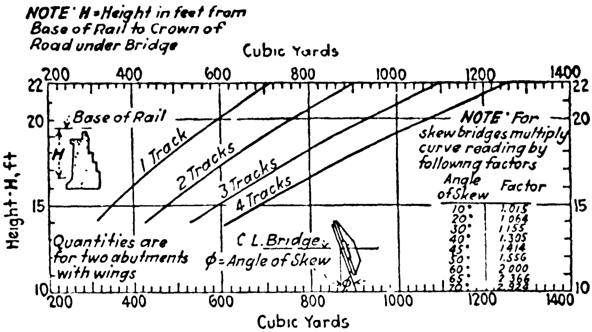


FIG. 216A.—Concrete in railroad bridge abutments N. Y. State 1926 standards.

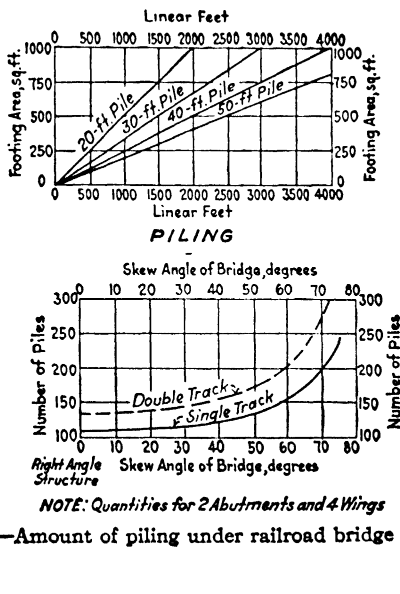


FIG. 217.—Amount of piling under railroad bridge abutments.

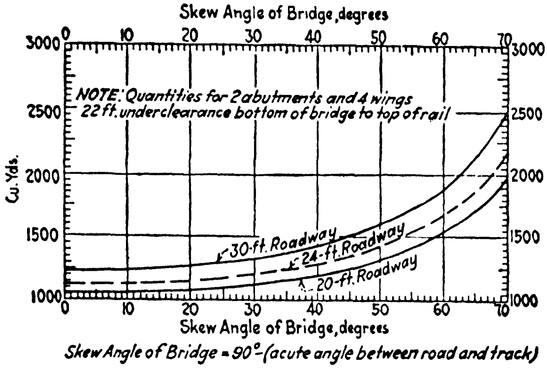


FIG. 218.—Quantities for highway bridge abutments overhead bridge eliminations.

PEDESTAL AND TRESTLE APPROACH DATA[1]

"Economical trestle span between bents:

16′ bent spacing (24′ roadway)	$130 cost per linear foot
20′ bent spacing (24′ roadway)	125 cost per linear foot
25′ bent spacing (24′ roadway)	135 cost per linear foot
30′ bent spacing (24′ roadway)	150 cost per linear foot

"NOTE.—Add 10 % for skew. Add $20 per foot for sidewalk.

"Unit prices:

1 : 2½ : 5 concrete foundations	$18 per cubic yard
1 : 2 : 4 reinforced concrete	30 per cubic yard
Reinforcing steel	6 cts. per pound
Piles	1.20 per foot

Per cent, reinforced pedestals, 0.3 (40 lb. per cubic yard).
Per cent, reinforced columns, 1.2 (160 lb. per cubic yard).
Per cent, reinforced beams and floors, 0.7 (100 lb. per cubic yard).
Per cent, reinforced T-beam construction, 1.4 (200 lb. per cubic yard).

"Modulus of rupture reinforced concrete (design stress), 400 lb.

"Allowable foundation pressures:

Ordinary clay and sand mixed	2 tons per square foot
Soft clay	1 ton per square foot
Stiff clay	4 tons per square foot
Gravel and sand	4 tons per square foot
Rock (poor)	5 tons per square foot
Rock (good)	25 tons per square foot

"Piles, 17-ton maximum per pile.
"Concrete weighs 4000 lb. per cubic yard.
"Conservative design, H-20 loading."

[1] Compiled by W. G. Harger, 1924.

Overhead Highway Bridge Pedestal Supports

"Quantities for four pedestals.
"Foundations (Ordinary soils 2 tons per square foot).
"Piles if used (17 tons per pile maximum).

50′ main span.......................... 48 piles minimum
75′ main span.......................... 56 piles minimum
100′ main span.......................... 68 piles minimum

"Four foundations 1:2½:5 concrete at $18 per cubic yard, steel reinforcement, quantities as follows:

	Concrete, cubic yards	Steel, pounds
50′ main span	50	2000
75′ main span	62	3000
100′ main span	74	4000

"Four columns 1:2:4 concrete at $25 per cubic yard (20′ seat to top of foundation, quantities as follows:

	Concrete, cubic yards	Steel, pounds
50′ main span	80	3200
75′ main span	86	3400
100′ main span	92	3700

"Load on each pedestal footing:

50′ main span............... 250,000 + 160,000 = 410,000
75′ main span............... 310,000 + 170,000 = 480,000
100′ main span............... 380,000 + 180,000 = 560,000

"Bottom dimensions of pedestal footings (ordinary soils):

	Area, square feet	Dimensions, feet
50′ main span	100	10 by 10
75′ main span	120	11 by 11
100′ main span	140	12 by 12

Superstructure (H-20 Loading)

(Quantities per foot of length, using 24′ roadway)

"Slab type (24′ roadway) as follows:

	Concrete, cubic yards	Steel reinforcement, pounds
16′ span between bents	1.67	168
20′ span between bents	2.05	202
25′ span between bents	2.42	250
Stiffener braces	0.20	20

"Reinforced concrete, stringer type (24′ roadway) as follows:

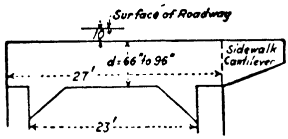
	Concrete, cubic yards	Steel reinforcement, pounds
16′ span between bents	1.89	380
20′ span between bents	2.08	420
25′ span between bents	2.44	500
30′ span between bents	2.89	580
Stiffener braces	0.1	20

"Six stringers spaced 5′ 06″.
"Ten-inch floor slab, stringers as follows:

16′ bents, 12″ wide, 42″ deep
20′ bents, 15″ wide, 42″ deep
25′ bents, 18″ wide, 48″ deep
30′ bents, 24″ wide, 48″ deep

"NOTE.—Where sidewalk is used add 15 % to above quantities. For H-15 loading reduce quantities 20 %.

TWO POST BENTS (CROSS-BEAMS AND BRACES)



Two-span bents	Concrete, cubic yards	Steel, pounds
16′ span between bents (24 by 78″)	9.7	1350
20′ span between bents (24 by 84″)	10.5	1450
25′ span between bents (30 by 90″)	13.0	1950
30′ span between bents (30 by 96″)	14.0	2100

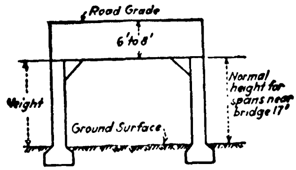
End bents	Concrete, cubic yards	Steel, pounds
16′ span (24 by 66″)	8.5	1150
20′ span (24 by 72″)	9.0	1250
25′ span (30 by 72″)	10.5	1600
30′ span (30 by 84″)	12.5	1850

"NOTES.—The quantities given (concrete) deduct stringers, that is, stringers and floor slab are computed by multiplying quantities by full length of approaches.

"If sidewalk is used add 15 % to these figures. For H-15 loading reduce quantities 20 %.

Two Post Bents

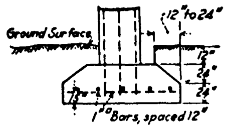
(Quantities for two posts per foot height between bottom of cross-beam and top of spread foundations)



Two-span bents	Concrete, cubic yards	Steel, pounds
16′ span (24 by 24″)	0.296	47
20′ span (24 by 24″)	0.296	47
25′ span (24 by 30″)	0.370	60
30′ span (24 by 30″)	0.370	60

End bents	Concrete, cubic yards	Steel, pounds
16′ span (24 by 24″)	0.296	47
20′ span (24 by 24″)	0.296	47
25′ span (24 by 30″)	0.370	60
30′ span (24 by 30″)	0.370	60

Two Post Bents (24′ Roadway)
Foundations



"Load on each footing.

	Two-span bents, pounds	End bents, pounds
16′ span	160,000	16′ span 125,000
20′ span	190,000	20′ span 145,000
25′ span	240,000	25′ span 180,000
30′ span	310,000	30′ span 225,000
"Piles if used for each column footing (20 to 30′ long):		
16′ span	5 piles	4 piles
20′ span	6 piles	5 piles
25′ span	7 piles	6 piles
30′ span	9 piles	7 piles

"Quantities for two-column footings (ordinary foundation soil, 2 tons per square foot):

	Two-span bents		End bents	
	Concrete, cubic yards	Steel, pounds	Concrete, cubic yards	Steel, pounds
16′ span...........	9.2	470	8.0	430
20′ span...........	12.1	560	9.2	470
25′ span...........	15.4	670	12.1	560
30′ span...........	20.0	800	13.7	610

"Bottom dimensions, column footings:

	Two-span bents, area, square feet		End bents, area, square feet	
16′ span........................	40	6 by 6	30	5 by 6
20′ span........................	50	7 by 7	35	6 by 6
25′ span........................	60	8 by 8	45	7 by 7
30′ span........................	80	9 by 9	55	7 by 8

UNIT PRICES BRIDGE AND GRADE-CROSSING WORK
Compiled, 1925, by W. G. Harger for Western New York Conditions

Item number	Item	Unit price
1	Clearing and grubbing	Special each case
2a	Earth excavation (borrow pit)	$ 0.70 per cubic yard
2b	Earth excavation roadway, common	0.90–1.50 per cubic yard
2c	Earth excavation foundations and culverts (dry)	2.00–2.50 per cubic yard
2d	Earth excavation foundations and culverts (wet)	4.00 per cubic yard
2e	Earth excavation slip scraper ditch work	0.75 per cubic yard
2f	Earth excavation sewers	Included in pipe price
2g	Backfill (abutments)	1.25 per cubic yard
3a	Rock excavation common roadway (shale)	2.00 per cubic yard
3aa	Rock excavation common roadway (hard)	2.50 per cubic yard
3b	Rock excavation in foundations (dry)	6.00 per cubic yard
3c	Rock excavation in foundations (wet)	8.00 per cubic yard
3d	Rock excavation removing old masonry	4.00 per cubic yard
5	Overhaul	0.007–0.01 per station yard
6	Sewer pipe	(See sewer diagrams)
7a	4″ underdrain	0.25 per linear foot
7b	6″ underdrain	0.35 per linear foot
10	Relaying old pipe	0.20 per linear foot
13a	12″ cast-iron pipe (medium weight)	3.50 per linear foot
13b	14″ cast-iron pipe (medium weight)	4.50 per linear foot
13c	16″ cast-iron pipe (medium weight)	5.70 per linear foot
13d	18″ cast-iron pipe (medium weight)	7.00 per linear foot
13e	20″ cast-iron pipe (medium weight)	8.50 per linear foot
13f	24″ cast-iron pipe (medium weight)	10.00 per linear foot
13g	30″ cast-iron pipe (medium weight)	15.00 per linear foot
13h	36″ cast-iron pipe (medium weight)	20.00 per linear foot
	Pointing old masonry	0.05 per square foot
16	Rip-rap	2.50 per cubic yard
17	Piles	0.90–1.20 per linear foot
18	Timber and lumber	80.00 per thousand
19	Portland cement	3.20 per barrel
20a	1:2:4 concrete[1] (jack arch floor)	18.00 per cubic yard
20b	1:2:4 concrete[1] (formed floor)	20.00 per cubic yard
20c	1:2:4 concrete[1] (slabs)	20.00 per cubic yard
20d	1:2:4 concrete[1] (trestle designs)	25.00 per cubic yard
21a	1:2½:5 concrete[1] foundations	10.00 per cubic yard
21b	1:2½:5 concrete[1] abutments (large work)	14.00 per cubic yard
21c	1:2½:5 concrete[1] abutments and pedestals (small)	16.00 per cubic yard
22a	1:3:6 concrete[1] pipe jackets	10.00 per cubic yard
27	Concrete[1] curbing	25.00 per cubic yard
27a	Curb bar	0.30 per linear foot
28	Concrete gutter (cement incl.)	18.00 per cubic yard
29	Cobble gutter (cement joints)	1.25 per square yard
30	Metal reinforcement concrete pavement	0.035 per square foot
31	Bar reinforcement concrete pavement	0.05 per pound
32b	Structural steel	
	Truss bridges	0.075 per pound
	Plate girders	0.07 per pound
	Rolled stringers	0.06 per pound

[1] Concrete prices do not include cement.

UNIT PRICES—*Continued*

Item number	Item	Unit price
32a	Bar steel (in structures)	$0.06 to $0.07 per pound
33	Miscellaneous iron and steel	0.10 per pound
33½	Corrugated iron for bridge floor jack arches	0.20 per square foot
34	Wooden guide rail	0.75 per foot
35	Cable guide rail	1.25 per foot
36	Concrete guide posts	3.25 each
37	2″ pipe railing	3.00 per foot
38	Preparing fine grade	0.10 per square yard
39	Run-of-bank gravel foundation	3.00 per cubic yard
41	Field or quarry-stone foundation	4.00 per cubic yard
42	Run-of-bank gravel bottom	3.50 per cubic yard
44	Broken-slag bottom	6.00 per cubic yard
45	Broken-stone bottom	7.00 per cubic yard
46	Concrete foundation for pavement (cement not included)	7.00 per cubic yard
47	Bituminous-macadam top (bitumen not included)	9.00 per cubic yard
51c	Concrete pavement (cement not included)	10.00 per cubic yard
54	Brick pavement*	3.50 per square yard
55	Stone-block pavement*	5.50 per square yard
56	Trimming shoulders	0.10 per linear foot of road
61	Broken slag (loose)	4.00 per cubic yard
62	Screened gravel (loose)	3.00–4.00 per cubic yard
63	Broken stone (loose)	4.50 per cubic yard
66	Bituminous material *A* (penetration)	0.15 per gallon
71	Bituminous material *T* (penetration)	0.17 per gallon
75	Maintaining traffic	0.20 per foot
76	Resetting wooden guide rail	0.50 per foot
	Waterproofing	0.10 per square foot

* Does not include concrete base.

Miscellaneous Items

Labor raising track	$ 0.80 per linear foot per foot raise.
Pile temporary track protection during construction of railway bridges	30.00 per foot per track.

Cost of Interurban Electric Track Work

Ballast, ties, poles and wiring	$5 per ft. single track.
Steel rails	$1.50 per ft. single track.

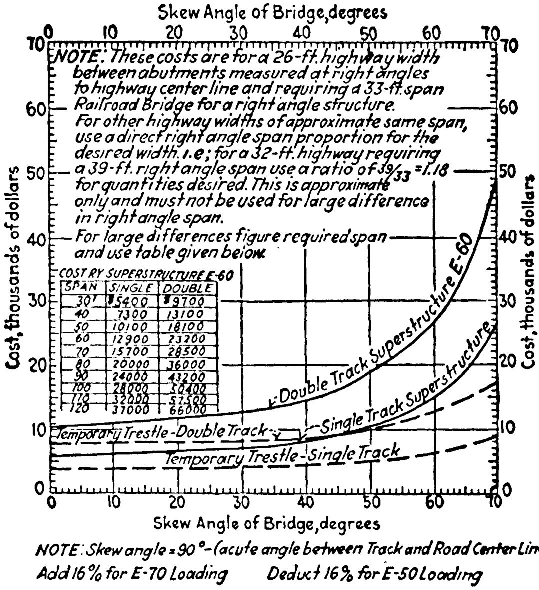


FIG. 219.—Cost diagrams railroad bridge superstructures.

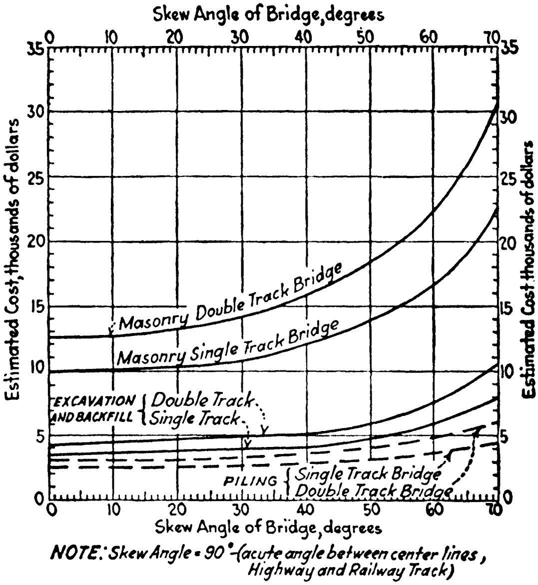
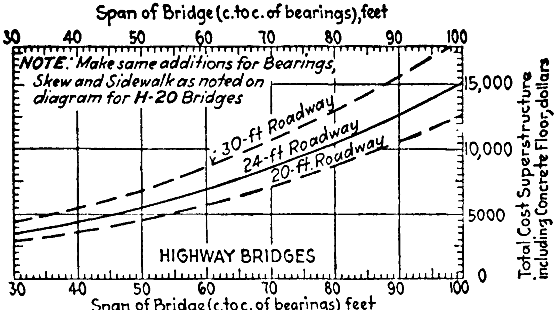
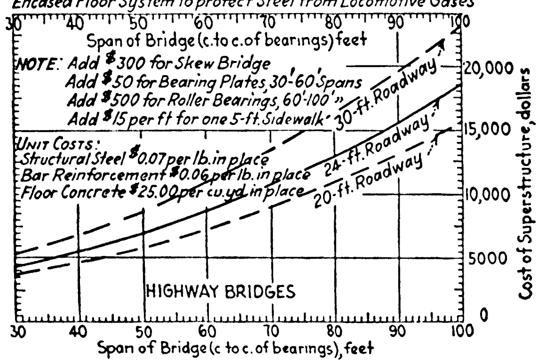


FIG. 220.—Approximate railroad bridge abutment costs.
Note: All costs are for 2 abutments and 4 wings unit costs 1925 conditions page 661.



Typical Plate Girder Cost Curves, H-15 Loading (15-Ton Trucks) Encased Floor System to protect Steel from Locomotive Gases



Typical Plate Girder Cost Curves, H-20 Loading (20-Ton Trucks) Floor Steel encased in Concrete to protect from Engine Gases

FIG. 221.—Typical cost curves plate girder highway bridge superstructures.

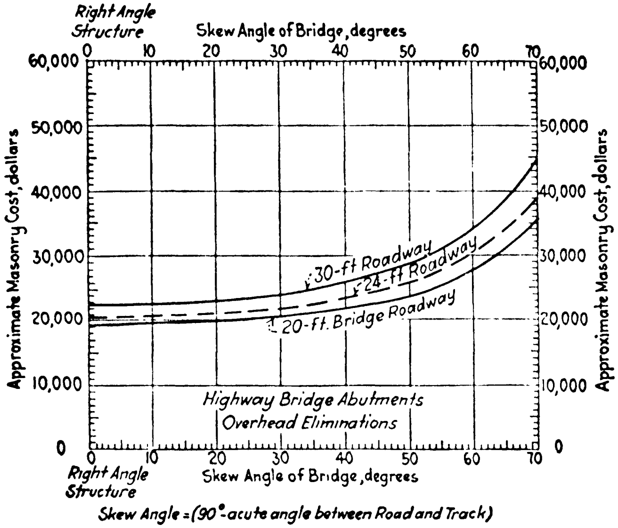


FIG. 222.—Approximate costs overhead highway bridge abutments. Masonry costs 2 abutments and 4 wings 22 ft. underclearance bottom of bridge to top of rail. Unit cost $18 per c. y. concrete. Figure exc. separately. Figure piles separately.

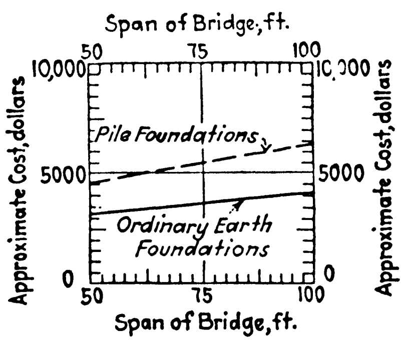


FIG. 223.—Approx. cost 4 pedestal supports. Overhead highway bridge H-20 loading (24′ roadway). Used in connection with trestle approaches. Excavation for 4 pedestals approx. $500.

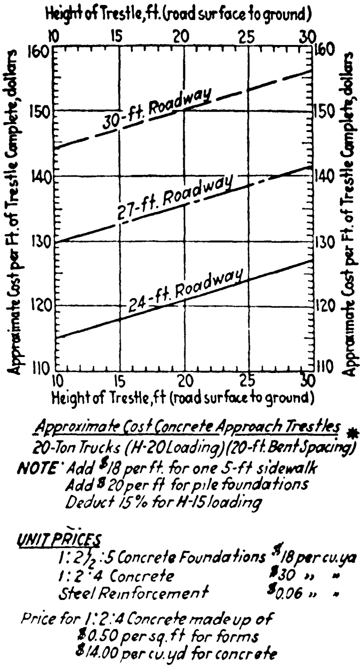


FIG. 224.—Typical costs highway concrete trestle approaches grade crossing eliminations.

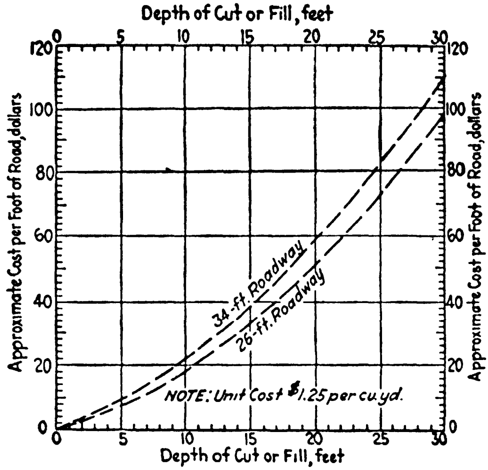


FIG. 225.—Typical costs highway earthwork on approaches grade crossing eliminations.

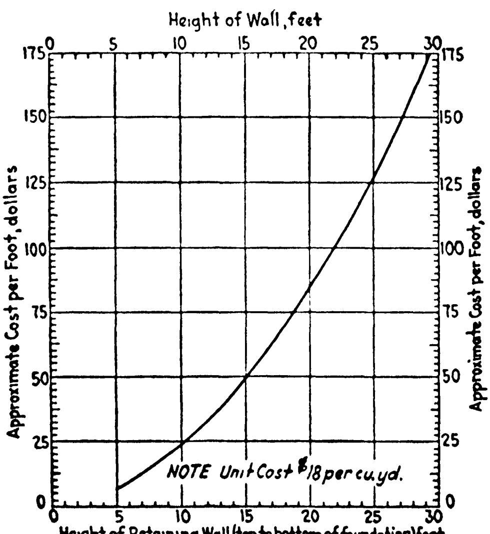


FIG. 226.—Typical cost curves gravity concrete retaining walls.

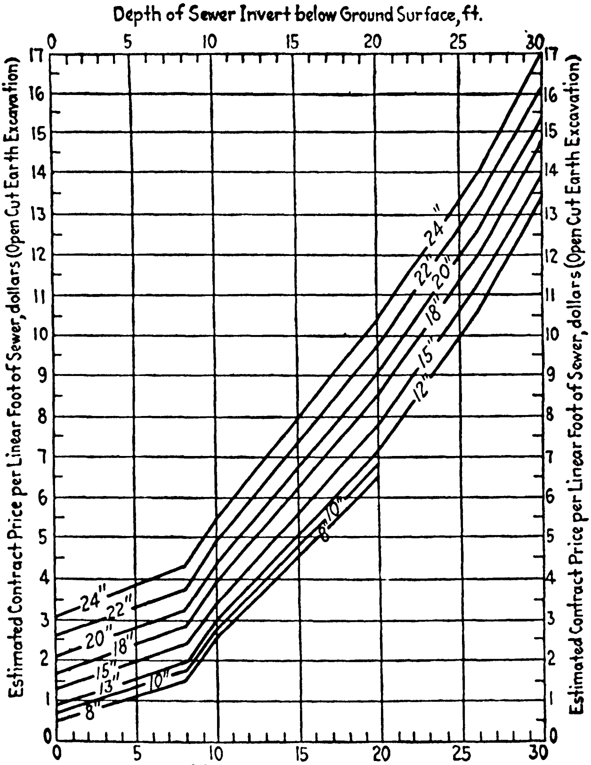


Fig. 227.—Typical cost of sewers including trench work as well as sewer proper (1922 cost conditions).

Graph No. 1 showing estimated contract costs per linear foot of vitrified pipe sewers at different depths.

Note.—These prices based on double strength pipe with cement joints.

If concrete jacket is used add item No. 55 to these prices.

If asphalt joints are used add item No. 19A to these prices.

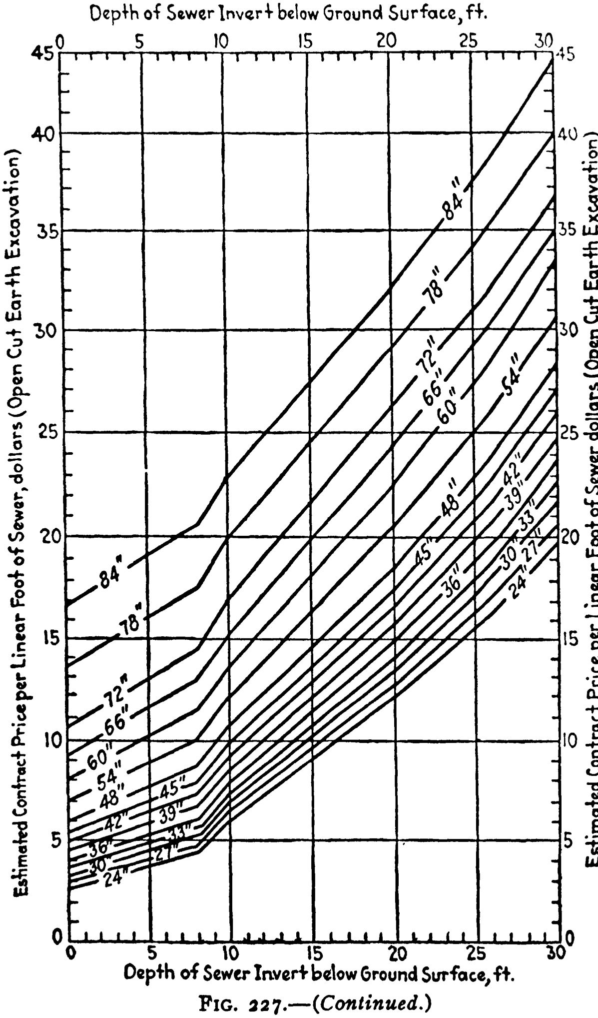


FIG. 227.—(*Continued.*)

Graph No. 2 showing estimated contract costs per linear foot of circular monolithic plain concrete sewers at different depths.

General Sewer Cost Data
City of Lima Ohio & Adjacent Territory
Prepared by
Fuller & McClintock
1921

Table Showing Minimum Amount of Class "B" Concrete Jacket per Lin. Ft. of Pipe Sewer

Size pipe, diameter in inches	Number of c. y. concrete jacket per lin. ft. of sewer
8	0.030
10	0.035
12	0.04
15	0.05
18	0.09
20	0.10
22	0.11
24	0.12

Table Showing Mininum Amount of Class "B" Concrete Cradle per Lin. Ft. of Pipe Sewer

Size pipe, diameter in inches	Number of c. y. concrete cradle per lin. ft. of sewer
27	0.32
30	0.34
36	0.39
42	0.45
48	0.50
54	0.57
60	0.63

Basic Data 1921 Scale of Prices

Common labor	$0.40 per hour
Hauling	0.30 per ton mile
Coal (f.o.b. cars Lima)	5.00 per ton
Cement (net bags returned)	2.50 per bbl.
Crushed stone (delivered on the work)	2.00 per c. y.
Concrete sand (f.o.b. cars Lima)	2.15 per ton
Sewer brick (f.o.b. cars Lima)	14.00 per M.
Timber for bracing	43.00 per M. ft. B.M.

Tile pipe

Size	Price
8	0.31
10	0.46
12	0.59
15	0.79
18	1.10
20	1.32
22	1.76
24	1.98

FIG. 227.—(*Continued.*)

FIG. 227.—(*Continued.*)

Item No.	List of items	Unit of measure	Estimated unit price
1–2½	Special concrete sewers	lin. ft.	Special
3–3A etc.	Special structures (junction chambers, etc.)	each	Special
(4–18)	Circular sewers 24″ to 84″ diam. (concrete or block)	lin. ft.	See Graph No. 2
4	24″ diameter circular sewers (open cut)	lin. ft.	See Graph No. 2
5	27″ diameter circular sewers	lin. ft.	See Graph No. 2
6	30″ diameter circular sewers	lin. ft.	See Graph No. 2
7	33″ diameter circular sewers	lin. ft.	See Graph No. 2
8	36″ diameter circular sewers	lin. ft.	See Graph No. 2
9	39″ diameter circular sewers	lin. ft.	See Graph No. 2
10	42″ diameter circular sewers	lin. ft.	See Graph No. 2
11	45″ diameter circular sewers	lin. ft.	See Graph No. 2
12	48″ diameter circular sewers	lin. ft.	See Graph No. 2
13	54″ diameter circular sewers	lin. ft.	See Graph No. 2
14	60″ diameter circular sewers	lin. ft.	See Graph No. 2
15	66″ diameter circular sewers	lin. ft.	See Graph No. 2
16	72″ diameter circular sewers	lin. ft.	See Graph No. 2
17	78″ diameter circular sewers	lin. ft.	See Graph No. 2
18	84″ diameter circular sewers	lin ft.	See Graph No. 2
(19–26)	Vitrified pipe sewers (cement joints), open cut	lin. ft.	Graph No. 1
19	8″ vitrified pipe sewer in open cut	lin. ft.	
19A	Bituminous joints in 8″ pipe sewer, extra	per joint	0.15
20	10″ vitrified pipe sewer in open cut	lin. ft.	Graph No. 1
20A	Bituminous joints in 10″ pipe sewer, extra	per joint	0.20
21	12″ vitrified pipe sewer in open cut	lin. ft.	Graph No. 1
21A	Bituminous joints in 12″ pipe sewer, extra	per joint	0.25
22	15″ vitrified pipe sewer in open cut	lin. ft.	Graph No. 1
22A	Bituminous joints in 15″ pipe sewer, extra	per joint	0.30
23	18″ vitrified pipe sewer in open cut	lin. ft.	Graph No. 1
23A	Bituminous joints in 18″ pipe sewer, extra	per joint	0.40
24	20″ vitrified pipe sewer in open cut	lin. ft.	Graph No. 1
24A	Bituminous joints in 20″ pipe sewer, extra	per joint	0.50
25	22″ vitrified pipe sewer in open cut	lin. ft.	Graph No. 1
25A	Bituminous joints in 22″ pipe sewer, extra	per joint	0.70
26	24″ vitrified pipe sewer in open cut	lin. ft.	Graph No. 1

26A	Bituminous joints in 24″ pipe sewer, extra	per joint	0.90
27	5″ and 6″ vit. pipe not otherwise provided for	lin. ft.	0.50
28	8″ vit. pipe not otherwise provided for	lin. ft.	0.60
29	10″ vit. pipe not otherwise provided for	lin. ft.	0.80
30	12″ vit. pipe not otherwise provided for	lin. ft.	1.00
31	15″ vit. pipe not otherwise provided for	lin. ft.	1.30
32	18″ vit. pipe not otherwise provided for	lin. ft.	1.70
33	20″ vit. pipe not otherwise provided for	lin. ft.	2.20
34	22″ vit. pipe not otherwise provided for	lin. ft.	2.60
35	24″ vit. pipe not otherwise provided for	lin. ft.	3.10
36	5″ and 6″ vit. pipe specials	each	1.20
37	8″ vit. pipe specials	each	1.50
38	10″ vit. pipe specials	each	1.70
39	12″ vit. pipe specials	each	2.00
40	15″ vit. pipe specials	each	2.80
41	18″ vit. pipe specials	each	3.60
42	20″ vit. pipe specials	each	4.60
43	22″ vit. pipe specials	each	5.50
44	24″ vit. pipe specials	each	7.00
45	6″ risers	lin. ft.	2.40
46	Standard manhole	lin. ft.	10.00
47	Rock excavation	c. y.	6.00
48	Additional earth excavation in open cut (0′–10′ deep)	c. y.	1.20
49	Additional earth excavation in open cut (10′–15′ deep)	c. y.	2.70
50	Additional earth excavation in open cut (15′–20′ deep)	c. y.	3.00
51	Additional earth excavation in open cut (20′–25′ deep)	c. y.	3.30
52	Additional earth excavation in open cut (25′–30′ deep)	c. y.	3.70
53	Additional earth excavation in open cut (30′–35′ deep)	c. y.	4.00
54	Additional earth excavation in open cut tunnel	c. y.	5.50
54½	Extra allowance for required tunnel on vit. pipe sewer	lin. ft.	1.00
55	Class "B" concrete jackets around vit. pipe sewers	c. y.	11.00
56	Additional class "B" concrete	c. y.	12.00
57	Additional class "A" concrete	c. y.	15.00
58	Additional brick masonry not otherwise provided for	c. y.	35.00
59	Sheeting and timbering left in place	c. y.	
60	Lumber and foundations		
61	Piles	lin. ft.	2.00
62	Iron castings	lb.	0.08

FIG. 227.—(*Continued.*)

Item No.	List of items	Unit of measure	Estimated unit price
62–01	Lock manhole covers	each	100.00
63	Steel reinforcement	lb.	0.08
64	Metal mesh reinforcement	lb.	0.10
65	Cast iron pipe	ton	100.00
66	Cast iron pipe specials	ton	200.00
67	Cutting and removing macadam pavements	sq. yd.	0.15
68	Cutting and removing block pavements on macadam base	sq. yd.	0.50
69	Cutting and removing pavements having concrete base	sq. yd.	1.20
70	Relaying block pavements on 9″ macadam base	sq. yd.	2.80
71	Relaying block pavements on 9″ class "B" concrete	sq. yd.	3.50
72	New 6″ waterbound macadam	sq. yd.	1.20
73	New 9″ waterbound macadam	sq. yd.	2.00
74	New 9″ bituminous macadam	sq. yd.	2.50
75	New 3″ bituminous macadam on 9″ concrete base	sq. yd.	3.50
76	New brick pavement on 9″ concrete base	sq. yd.	5.00
77	New asphalt block pavement on 9″ concrete base	sq. yd.	5.50
78	New wood block pavement on 9″ concrete base	sq. yd.	6.00
79	New sheet asphalt pavement on 9″ concrete base	sq. yd.	4.50
80	New reinforced cement concrete pavement 9″ thick	sq. yd.	4.20
81	Special tamping	c. y.	1.00
82	Tamping up and relaying sidewalk	sq. ft.	0.25
83	Removing and resetting or rebuilding curb and gutter	lin. ft.	1.50
84	Catch basins for combined sewers (trapped)	each	70.00
85	Catch basins for separate storm sewers	each	50.00
86	Track protection	l. s.	Special
87	Maintaining existing flow of sewage	l. s.	Special
88	Care of existing structures	l. s.	Special
89	Loose stone for temporary road surface	c. y.	Special
90	Special items		
91			
92			
93			
etc.			

FIG. 227.—(*Continued.*)

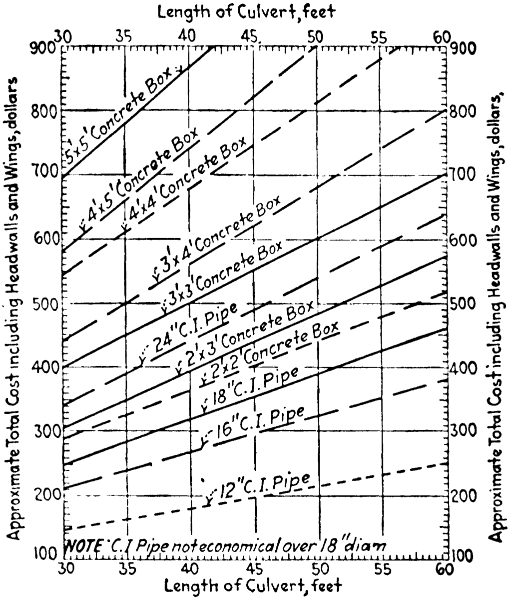


FIG. 228.—Approx. cost of culverts (exclusive of excavation) N. Y. State standards.

Note: Cast iron pipe not economical over 18" dia.

CHAPTER X

MINOR POINTS OF DESIGN

For convenience of reference this chapter gives a few illustrations of current practice in connection with incidental features of road construction, such as guide rails, retaining walls, curbs, sidewalks, cobble ditch protection, guide and danger signs, snow fences, etc. These data are perhaps unnecessary but incidental information is useful in emergencies and gives the student some idea of current practice in minor details.

Guard Rail.—The guard rail serves two purposes, to warn and to protect. If protection is essential at really dangerous points a

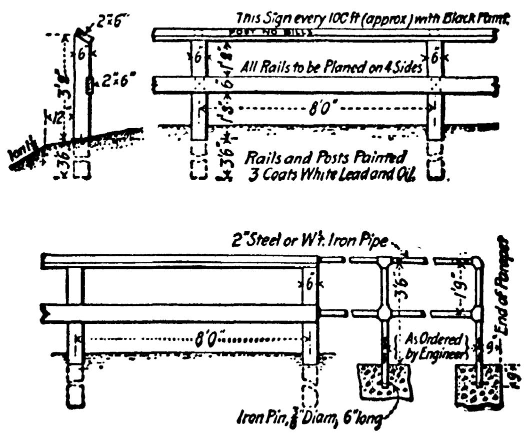


FIG. 229.—Wooden guide rail (New York State 1921). This type is a warning rail only.

solid retaining wall is desirable. Most types of so-called guard rails serve more in the nature of a warning than as an actual protection. Wooden structures require continual maintenance, and for this reason concrete posts, rail, and steel cables have been used on the score of reducing the necessity of continual repair. Figures 229 to 233 show usual types.

Retaining Walls.—In unusual cases, retaining walls are needed in road construction. Plain or reinforced-concrete walls are generally used, the selection depending on the relative cost. The plain concrete wall is considered the best type for heights up to 12′; the reinforced cantilever form from 12 to 18′; and above 18′, the buttressed design. Standard practice for the plain and reinforced cantilever types page 682. The necessity for walls higher than 18′ is very rare. For the design of butressed walls the reader is referred to the standard works on reinforced concrete.

Retaining walls are usually built in monolithic sections of 20 to 25′ in length; expansion joints are provided between these sections. The expansion joints may consist simply of a plane of weakness between the sections, produced by allowing one section

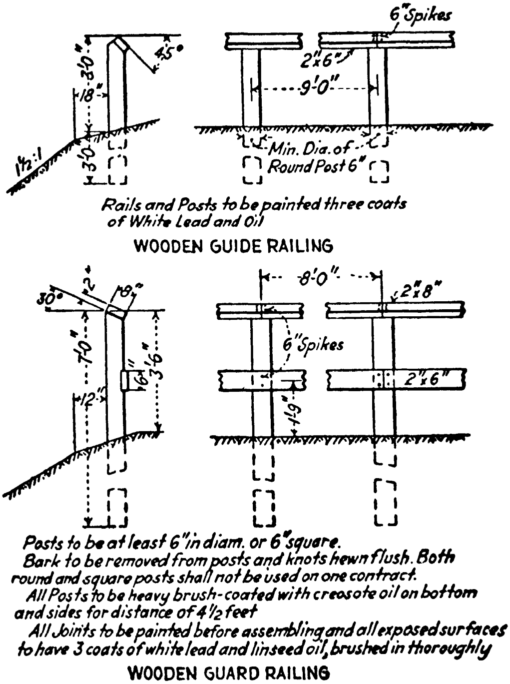
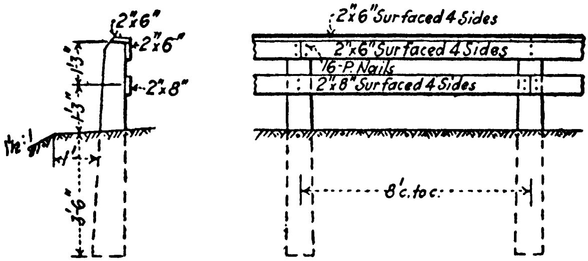


FIG. 229A.—Wooden guide rail U. S. Bureau Public Roads 1926.

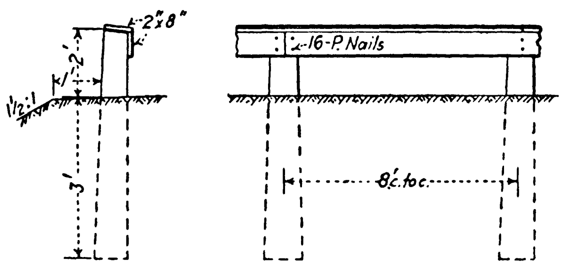
to set before building the adjacent wall, or it may be a key joint as shown in Fig. 236, and the plane of separation may be made more pronounced by coating the concrete with a thin layer of asphaltum or pitch (Fig. 236A).

Repointing Masonry and Refacing Old Walls.—Old masonry structures can often be used complete or in part by repointing the joints; they should be cleaned out thoroughly with a chisel and filled flush with 1:1 Portland-cement mortar. The old joints

(text continued on page 681.)



WOODEN GUARD RAIL WITH HUB GUARD PLANK



Posts to be at least 6" in diam or 6" square
Bark to be removed from posts and knots hewn flush, both round and square posts shall not be used on one contract
All Posts to be heavy brush-coated with creosote oil on bottom and sides up to a point 8" above ground level
All Joints to be painted before assembling and all exposed surfaces to have 3 coats of white lead and linseed oil, brushed in thoroughly

WOODEN HUB-HIGH GUARD RAIL

FIG. 229*B*.—Typical wooden hub guard. U. S. Bureau Public Roads 1926.

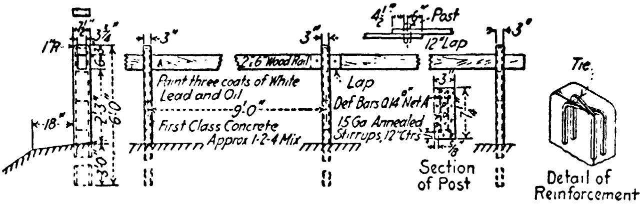


FIG. 230.—Concrete-wooden guide rail (New York State 1922). This type is a warning rail only.

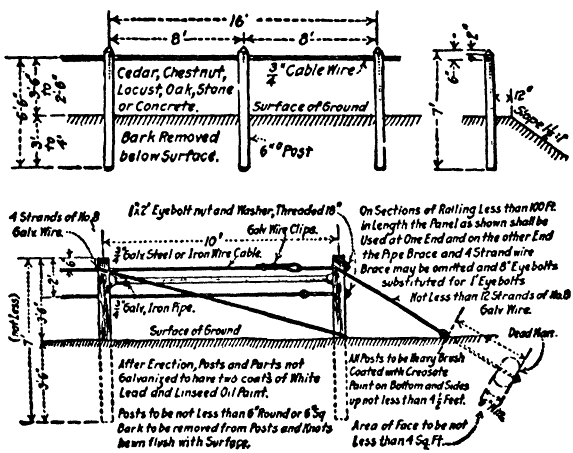


FIG. 231.—Cable guard rail (Penn. 1924). This type gives some protection.

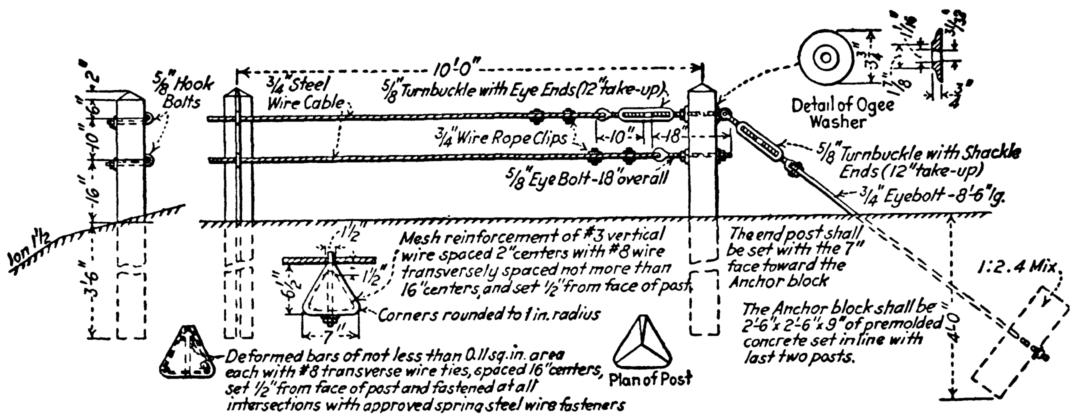


FIG. 232*A*.—Steel cable rail with concrete posts (N. Y. State 1927).

NOTE.—These same posts without cable are used as guide posts. They are a cheap and effective protection on straight alignment when spaced from 6 to 12 ft. between posts.

should be well cleaned out and hook dowels used as shown in Fig. 237. One dowel every 6 sq. ft. is good practice.

The concrete facing should be at least 12″ thick, have a good footing course, and be reinforced to prevent settlement and temperature cracks.

Toe Walls.—Toe walls are nothing more than low retaining walls or very substantial curbs. They are used in cuts on the outside of the gutters to prevent unstable side slopes from filling the gutters or heaving them out of shape by sliding pressure. Figure 238 gives a section of Eden Valley Hill near Buffalo, N. Y., where a clay quicksand cut was successfully protected in this manner.

Curbs.—Curbs are constructed of stone and of concrete (Fig. 239).

Stone Curbs.—The cuts given show the methods of setting; the size of curbstones for first-class work range from 16 to 22″ in depth, 5 to 6″ in thickness, and 3 to 5′ in length. For small villages, curbstone of 4″ width set in the simplest manner shown is satisfactory. The stones most used are granites, bluestones of

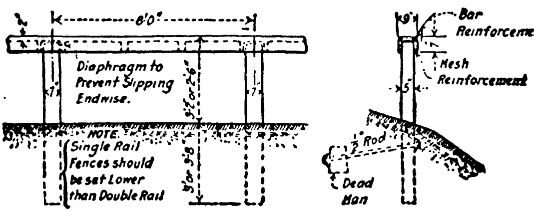


FIG. 233.—Typical reinforced concrete rail with post anchor blocks. This type gives some real protection as well as acting as a warning.

New York State, and the tougher sandstones, such as Medina, Berea, Kettle River, etc. The prices range widely, depending on the locality of the work.

Curb Radii.—A good radius for drives is 4′; for right-angle main-street intersections, 13′. For acute or obtuse angles 10 to 20′. Figure 240 shows a satisfatory sidewalk corner used in Rochester, N. Y. Where the street width and sidewalk layout permit a longer curb radius without inconvenience to pedestrians, the present tendency is to increase the curb radius to 25′. This permits the ordinary car to make the turn easily without swinging out from the curb traffic lane. This applies mainly to side streets intersecting with congested main streets.

Cobble Gutters, Brick Gutters, Ditch Linings, Etc.—Cobble gutters are used to protect the ditches from wash on steep grades and at entrances to intersecting roads where there is not sufficient headroom for a culvert, also at the entrances to private property where the grade line of the ditch might be badly cut by vehicles. Where cobblestones are not available, ordinary building brick may be used, or No. 4 crushed stone as shown in Fig. 241.

(text continued on page 687.)

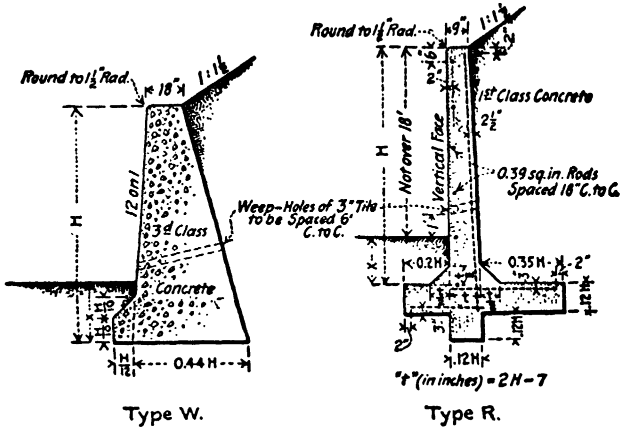


FIG. 234.—New York State standard retaining walls.

TABLE 116

H	Reinforcing Steel Bars of Deformed Section								
	Stem[1]			Heel			Toe		
	Net Area	Spacing C–C	Length	Net Area	Spacing C–C	Length	Net Area	Spacing C–C	Length
11′	0.601	6¼″	12′–2″	0.442	7½″	4′–11½″	0.442	9½″	3′–3½″
12′	0.601	5½″	13′–3½″	0.442	6¼″	5′–5½″	0.442	8¼″	3–8″
13′	0.601	5″	14′–5″	0.442	5¼″	6′–0″	0.442	7¼″	4′–0½″
14′	0.601	4½″	15′–6½″	0.601	6¼″	6′–6″	0.601	8½″	4′–4½″
15′	0.601	4″	16′–8″	0.601	5⅜″	7′–0″	0.601	7⅜″	4′–9″
16′	0.994	6″	17′–9″	0.601	4¾″	7′–6½″	0.601	6½″	5′–1½″
17′	0.994	5½″	18′–10½″	0.785	5½″	8′–0½″	0.785	7⅝″	5′–6″
18′	0.994	5″	20′–0″	0.785	4⅞″	8′–7″	0.785	6⅞″	5′–10½″
19′	0.994	4½″	21′–1½″	0.785	4⅜″	9′–1″	0.785	6¼″	6′–2½″
20′	0.994	4⅛″	22′–3″	0.785	4″	9′–7″	0.785	5¾″	6′–7″

[1] In each set of 3 bars in stem, first bar which is of length given, extends to top of wall, second bar to height ⅔ H, third bar to height ⅓ H.

When Type W is used as a bank wall (that is, above the roadway), max. $H = 20'$; min. $X = 2'$ for H of 5 to 10′; and 0.2 H for H greater than 10′.

When Type W is used as a sustaining wall (that is, below the roadway), max. $H = 13'$; and min. $X = 3'$, except where foundation is rock or entirely below frost.

When Type R is used as a bank wall, max. $H = 20'$; min. $X = 0.15\ H$ for H greater than 10′.

When Type R is used as a sustaining wall, max. $H = 13'$; min. $X = 0.25\ H$ for H greater than 10′.

TABLE 117.—RETAINING WALLS, STATE OF PENNSYLVANIA

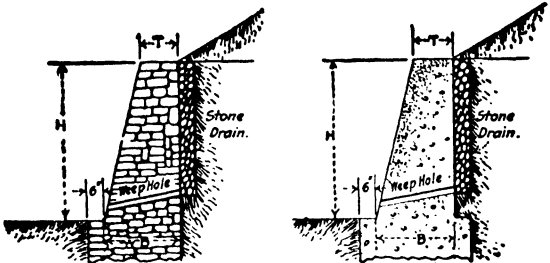
Cement Masonry						Dry Rubble Masonry						Concrete Masonry					
Ground Level with Top of Wall			Surcharged			Ground Level with Top of Wall			Surcharged			Ground Level with Top of Wall			Surcharged		
H.	T.	B.	H.	T.	B.	H.	T.	B.	H.	T.	B.	H.	T.	B.	H.	T.	B.
4'-0"	1'-6"	1'-8"	4'-0"	2'-0"	2'-3"	4'-0"	2'-0"	2'-0"	4'-0"	2'-0"	2'-7"	4'-0"	1'-6"	1'-6"	4'-0"	1'-6"	2'-0"
5'-0"	2'-0"	2'-0"	5'-0"	2'-0"	2'-10"	5'-0"	2'-0"	2'-6"	5'-0"	2'-0"	3'-4"	5'-0"	1'-6"	1'-9"	5'-0"	1'-6"	2'-5"
6'-0"	2'-0"	2'-5"	6'-0"	2'-0"	3'-5"	6'-0"	2'-0"	3'-0"	6'-0"	2'-0"	4'-0"	6'-0"	1'-6"	2'-2"	6'-0"	1'-6"	2'-10"
7'-0"	2'-0"	2'-10"	7'-0"	2'-0"	3'-10"	7'-0"	2'-0"	3'-6"	7'-0"	2'-0"	4'-7"	7'-0"	1'-6"	2'-5"	7'-0"	1'-6"	3'-5"
8'-0"	2'-0"	3'-3"	8'-0"	2'-0"	4'-6"	8'-0"	2'-0"	4'-0"	8'-0"	2'-0"	5'-4"	8'-0"	1'-6"	2'-10"	8'-0"	1'-6"	3'-10"
9'-0"	2'-0"	3'-7"	9'-0"	2'-0"	5'-0"	9'-0"	2'-0"	4'-6"	9'-0"	2'-0"	5'-10"	9'-0"	1'-6"	3'-2"	9'-0"	1'-6"	4'-4"
10'-0"	2'-0"	4'-0"	10'-0"	2'-0"	5'-7"	10'-0"	2'-0"	5'-0"	10'-0"	2'-0"	6'-7"	10'-0"	1'-6"	3'-6"	10'-0"	1'-6"	4'-10"

Cement.—**Top of wall not less than 1'–6", 2'–0" if possible.** Face battered; back perpendicular. Weep holes with blind drain in wet localities.

Dry Rubble.—**Top** of wall not less than 2'–0". Face battered; back perpendicular. Courses perpendicular to face batter **Face** of wall pointed. Weep holes with blind drain in wet localities.

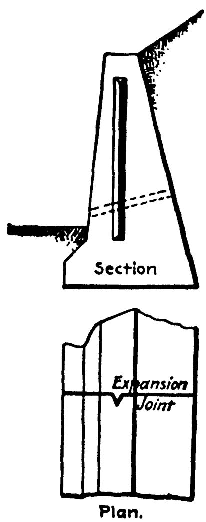
Concrete.—Top of wall not less than 1'–6". Face battered 1: 12; back battered or stepped. Weep holes with blind drain in wet localities. Expansion joints every 25'–0".

All offsets 6". Bottom of footing below frost line. (Depth of footing at least 3'.) Backfilling done with acceptable material placed in layers of not less than 6" and thoroughly rammed. (See page 684 for Typical Sections.)



Cement and Dry Rubble. Concrete

FIG. 235.—State of Pennsylvania retaining walls.
(To accompany Table 117 page 683.)



Key Expansion Joint.

FIG. 236A.

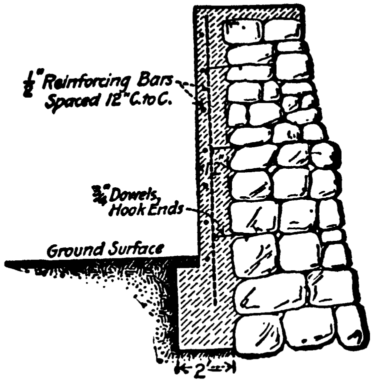


FIG. 237.—Facing old masonry.

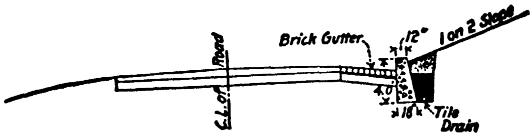


FIG. 238.—Typical toe wall.

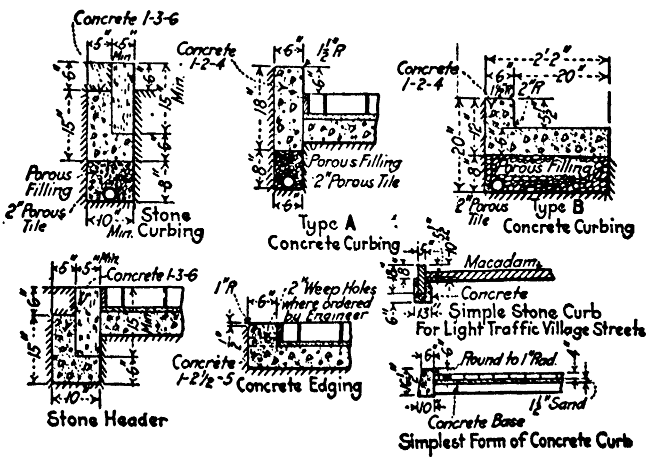


FIG. 239.—Typical curb details.

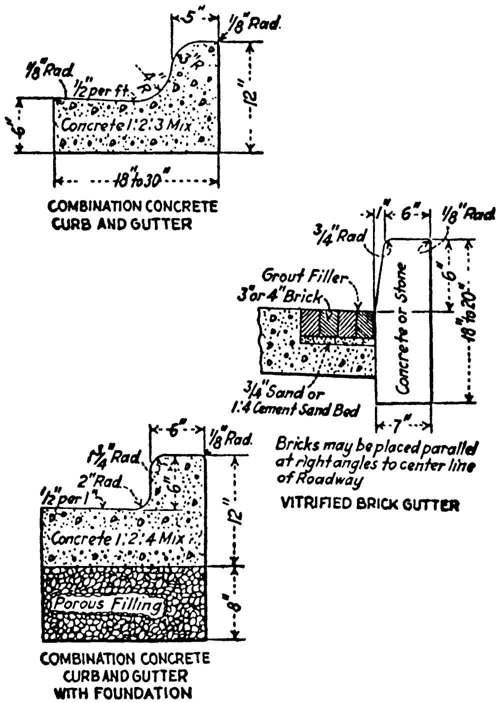


FIG. 239A.—Typical curb details U. S. Bureau Public Roads (1926).

Concrete gutters are in common use but are not so satisfactory as the cobble or brick gutter except on short hills where the gutter is directly adjacent to a concrete pavement.

Catch Basins.—Figure 242 shows ordinary rural highway catch basins and drop inlets. Figure 243 shows a satisfactory curb inlet city or village catch basin. Catch basins are to be avoided on rural highways if possible as the grate tends to clog and cause ponding. Where they are necessary, some clear opening not entirely covered by the grate should be provided.

Guide and Danger Signs.—Signs add to the convenience and safety of travel. Good examples of standard practice are shown in Figs. 252 and 197. The indiscriminate use of danger signs is inadvisable. Their use should be confined to actual danger condi-

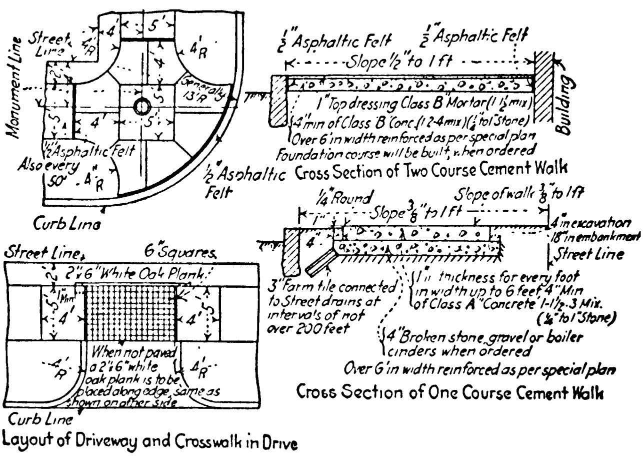


FIG. 240.—Typical sidewalk detail and curb corner layout (Rochester, N. Y.).

tions and they should be placed so that headlights at night will show them up easily. This last requirement generally results in placing them about 3 to 4′ from the edge of the pavement on the right side and not over 3′ above ground and preferably about 18″ above the shoulder to the bottom of the sign. The height and location must be worked out for each case and depend on alignment and grades.

The matter of location and height of signs is entitled to careful study for each individual case. There are, however, certain well-defined principles of location and height. The caution or danger sign should be located far enough from the point of danger to warn the driver in time for him to get his car under perfect control. This requires approximately 500′ on steep down grades, 250′ steep up grades, and about 300 to 350′ on ordinary light grades. The distance from the danger point should not be made any greater than

(text continued on page 689.)

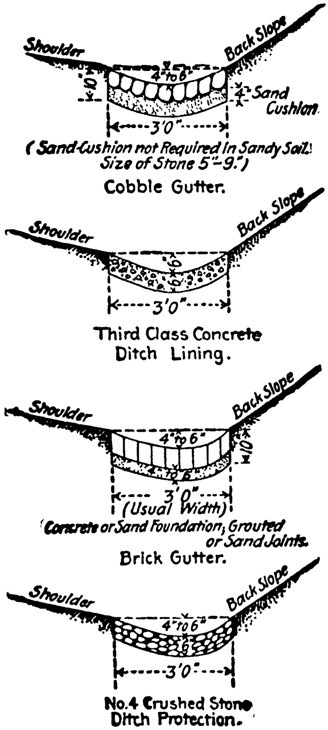


FIG. 241.—Typical ditch protections.

necessary, however. It is well established that the sign should be on the right side of the road and as near the pavement as possible without danger of being hit. Just back of the curb line on streets and from 3 to 4′ off the edge of pavement on rural highways are good locations. At dead end roads or very sharp turns it is desirable to place an additional large danger sign beyond the turn directly in line with the approach center line.

On the height of the sign depends the ease with which it is seen both day and night. In the daytime a distinctive sign can be readily seen at any elevation between 1 and 6′ above the road. The height is controlled, therefore, by night driving conditions, which

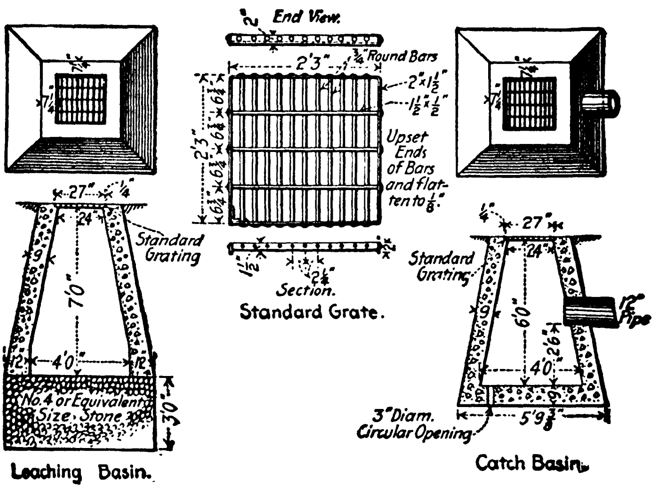


FIG. 242.

require a low position for proper headlight illumination. During the summer months when there is no snow on the ground, the best height is approximately 18″ from ground elevation to the bottom of the sign. In winter the reflection on the snow permits a height of 36″. Drifting snow makes the 36″ height desirable. For northern climates the danger signs should be of adjustable height, the lower height being used in summer and the higher in winter.

Figure 197 shows good typical locations and heights.

Signs should be uniform in color, shape, and symbols. The following quotation gives the most recent recommendations for uniform signals (1926).

(text continued on page 697.)

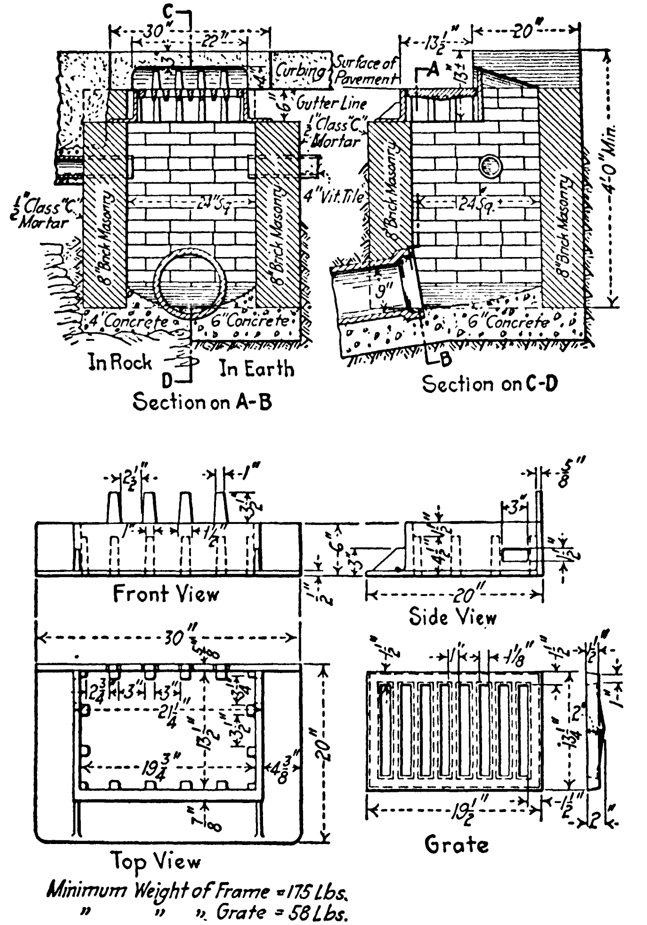
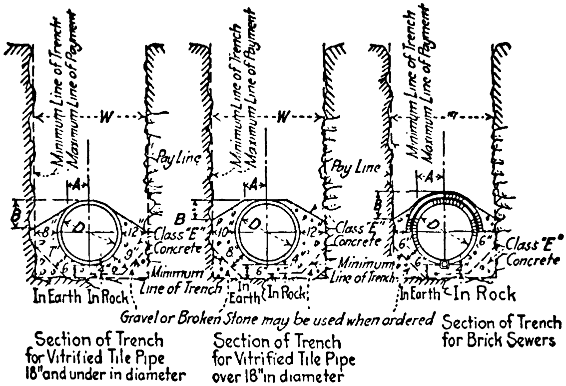


FIG. 243.—Typical curb inlet catch basin (City of Rochester Standard).



Classification for Concrete					
Class	A	B	C	D	E
Prop	1:1 1/2:3	1.2:4	1:2 1/2:5	1.3:6	1:8
Aggregate	1/4" to 1" Stone	1/4" to 1" Stone	1/4" to 1 1/2" Stone	1/4" to 2" Stone	Torpedo Gravel

Classification for Mortar				
Class	A	B	C	D
Prop	1:1	1:1 1/2	1:2 1/2	1:3

Pipe Sewers 18" and Under							
In Earth				In Rock			
D	W	A	B	D	W	A	B
6"	23 1/4"	2 5/8"	6 1/4"	6"	31 1/4"	2 5/8"	8 5/8"
8"	25 1/2"	3 3/8"	6 3/4"	8"	33 1/2"	3 3/8"	9 1/8"
9"	26 3/4"	3 3/4"	7 1/8"	9"	34 3/4"	3 3/4"	9 3/8"
10"	27 3/4"	4 1/8"	7 3/8"	10"	35 3/4"	4 1/8"	9 5/8"
12"	30"	5"	7 7/8"	12"	38"	5"	10 1/8"
15"	33 1/2"	6 1/4"	8 3/4"	15"	41 1/2"	6 1/4"	11"
18"	37"	7 3/8"	9 3/8"	18"	45"	7 3/8"	11 7/8"

Pipe Sewers Over 18"							
In Earth				In Rock			
D	W	A	B	D	W	A	B
20"	43 1/2	7 1/2"	8 1/4"	20"	47 1/2"	7 1/2"	9 3/8"
22"	45 3/8"	8 1/4"	8 1/2"	22"	49 5/8"	8 1/4"	9 5/8"
24"	48"	9"	8 5/8"	24"	52"	9"	9 7/8"
27"	51 5/8"	10 1/4"	9 1/8"	27"	55 1/2"	10 1/4"	10 1/4"
30"	55 7/8	11 1/4"	9 3/8"	30"	59"	11 1/4"	10 5/8"

Brick Sewer 30" and Over in Rock and Earth			
D	W	A	B
30"	4'-11"	16 1/2"	14 3/8
33"	5'-2"	17 5/8"	15"
36"	5'-5"	18 1/4"	15 5/8"
42"	5'-11"	20 3/8"	17"
45"	6'-2"	21 7/8"	17 3/4"
48"	6'-5"	23"	18 5/8"

FIG. 244.—Sewer trenches.

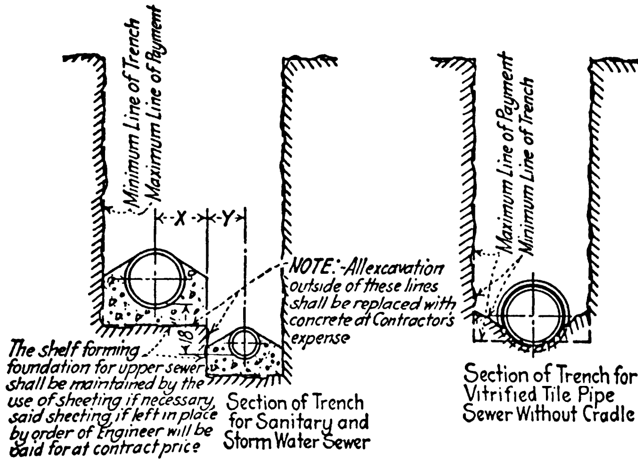


Table Showing Distances X and Y for Sewers laid in the same Trench, the Distance between center lines of Pipes = X+Y.
The total width of Trench paid for and the minimum excavation width = 2X+2Y

Kind	Size	In Earth		In Rock		Kind	Size	In Earth		In Rock	
Tile Pipe		X	Y	X	Y	Tile Pipe		X	Y	X	Y
" "	6"	11 5/8"	11 5/8"	15 3/4"	15 3/4"	" "	24"	24"	24"	26"	26"
" "	8"	12 3/4"	12 3/4"	16 7/8"	16 7/8"	" "	27"	25 3/4"	25 3/4"	27 3/4"	27 3/4"
" "	9"	13 3/8"	13 3/8"	17 3/8"	17 3/8"	" "	30"	28"	28"	29 1/2"	29 1/2"
" "	10"	13 7/8"	13 7/8"	17 5/8"	17 5/8"	Brick	30"	2'-5 1/2"	2'-5 1/2"	2'-5 1/2"	2'-5 1/2"
" "	12"	15"	15"	19"	19"	"	33"	2'-7"	2'-7"	2'-7"	2'-7"
" "	15"	16 3/4"	16 3/4"	20 3/4"	20 3/4"	"	36"	2'-8 3/8"	2'-8 3/8"	2'-8 3/8"	2'-8 3/8"
" "	18"	18 1/2"	18 1/2"	22 1/2"	22 1/2"	"	42"	2'-11 1/2"	2'-11 1/2"	2'-11 1/2"	2'-11 1/2"
" "	20"	21 3/4"	21 3/4"	23 3/4"	23 3/4"	"	45"	3'-1"	3'-1"	3'-1"	3'-1"
" "	22"	23"	23"	24 3/4"	24 3/4"	"	48"	3'-2 1/2"	3'-2 1/2"	3'-2 1/2"	3'-2 1/2"

FIG. 245.—Miscellaneous street standards (City of Rochester). Sewer trenches.

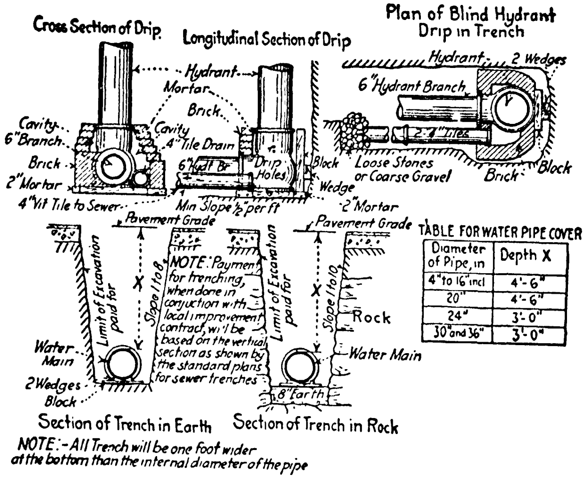
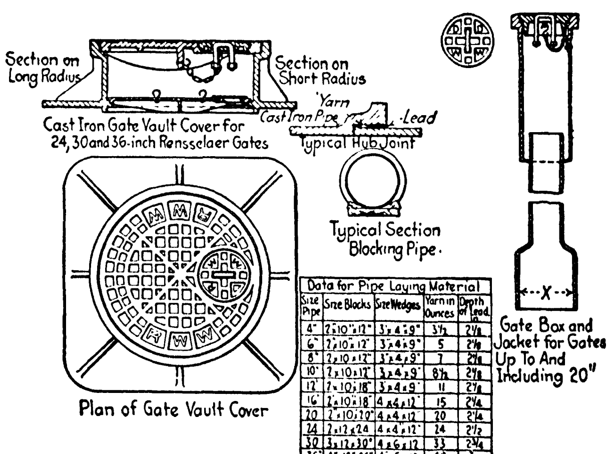


TABLE FOR WATER PIPE COVER

Diameter of Pipe, in	Depth X
4" to 16" incl	4'-6"
20"	4'-6"
24"	3'-0"
30" and 36"	3'-0"

FIG. 246.—Water pipe trenches.



Data for Pipe Laying Material

Size Pipe	Size Blocks	Size Wedges	Yarn in Ounces	Depth of Lead in
4"	2"x10"x12"	3"x4"x9"	3½	2⅛
6"	2"x10"x12"	3"x4"x9"	5	2⅛
8"	2x10x12"	3"x4x9"	7	2⅛
10"	2x10x12"	3x4x9"	8½	2⅛
12"	2x10x18"	3x4x9"	11	2⅛
16"	2"x10"x18"	4x4x12"	15	2¼
20	2"x10x20"	4x4x12"	20	2¼
24	2x12x24	4x4"x12"	24	2½
30	3x12x30"	4x6x12	33	2¾
36"	4"x12"x36"	4"x6x12	48	3

FIG. 247.—Miscellaneous street standards (City of Rochester). Water trenches, etc.

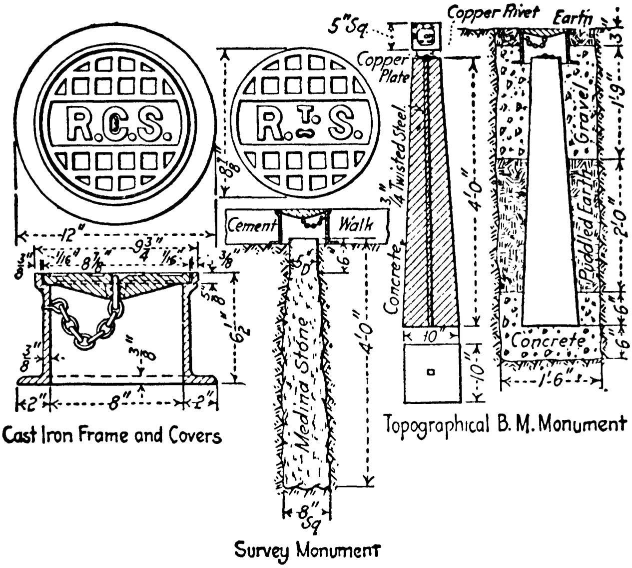


FIG. 248.—Miscellaneous street standards (City of Rochester).

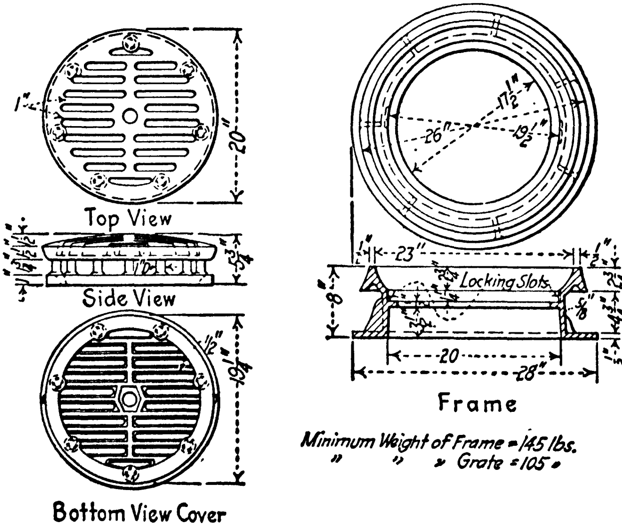


FIG. 249.—Miscellaneous street standards (City of Rochester).

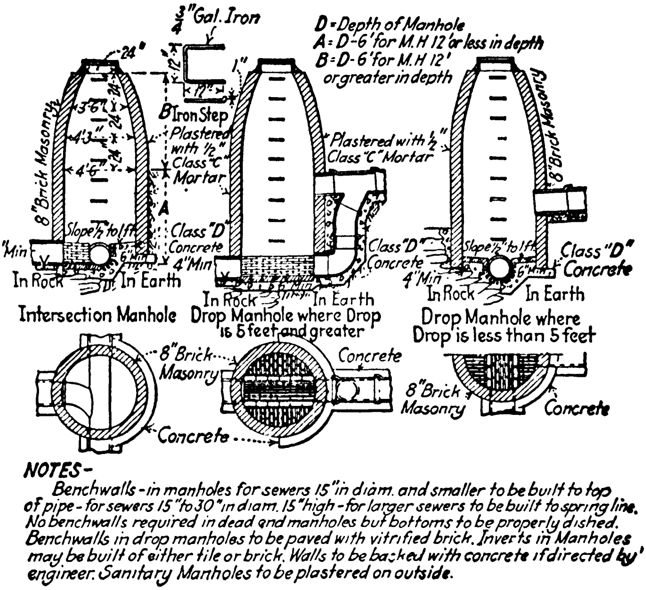


FIG. 250.—Miscellaneous street standards (City of Rochester). Manholes.

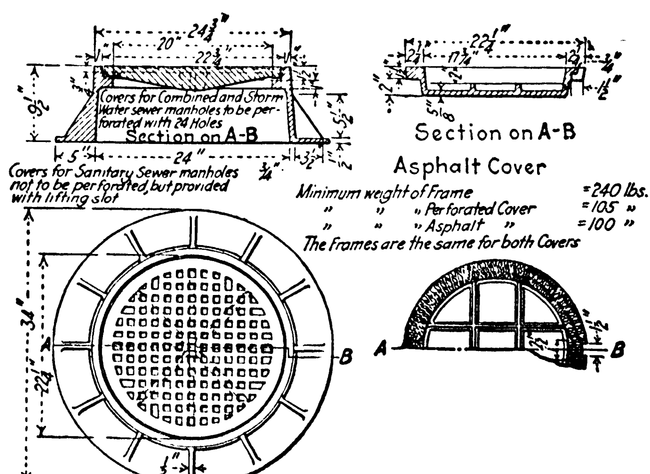


FIG. 251.—Miscellaneous street standards (City of Rochester). Manhole covers.

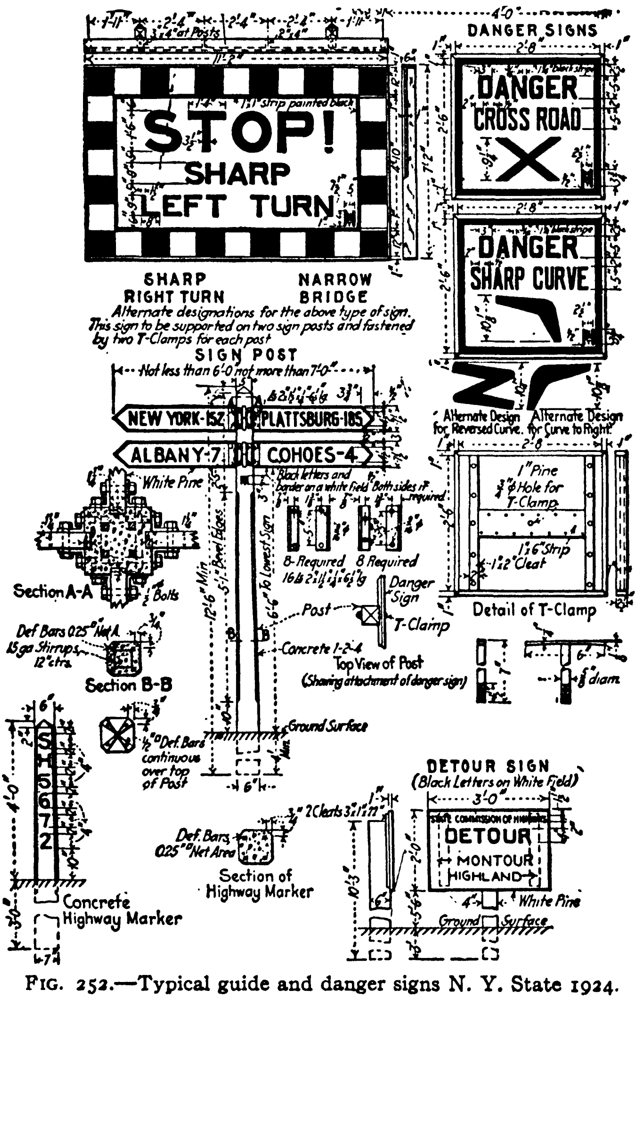


FIG. 252.—Typical guide and danger signs N. Y. State 1924.

HIGHWAY SIGNS AND SIGNALS

Recommendations of Subcommittee on Traffic Control and Safety Presented at San Francisco Meeting of American Association of State Highway Officials

Color of Signs.—Color of illuminated highway signs and signals.

RED to indicate STOP.
YELLOW to indicate CAUTION.
GREEN to indicate GO.

"These are the colors most generally used in railroad operations and are now quite generally used in highway work. It is believed that they should be universally adopted in regulation of traffic on the highway.

COLOR OF NON-LUMINOUS SIGNS

Sign	Background	Copy[1]
Stop	Red center panel White upper and lower panels	White Black
Caution	Yellow	Black
Go or safety	Green	White
Road markers, guide, direction, general information	White	Black

[1] Owing to the lack of visibility of red in non-luminous signs at night, the upper and lower panels are made white which, because of the contrast, will make a very conspicuous sign and at the same time retain the red which should be universally used for Stop.

"The first three colors are the same for non-luminous signs as for the luminous signs. It is believed that by keeping these colors the same, most confusion can be avoided. Because of their simplicity and agreeing with the luminous signs, it will be an easy matter to impress the color scheme clearly upon the minds of drivers, and even school children. No confusion should result with this color scheme universally adopted.

"**Shapes of Non-luminous Signs.**—While the color on the non-luminous signs will be important, it is believed that a variation in shapes, varying with the different degrees and kinds of danger and information will be most effective. It is a well-known fact that the shape of a sign is distinguishable long before the copy or symbols thereon can be recognized; hence, the importance of the variable shapes. In order to simplify and reduce cost of production it is necessary that these shapes be such that they can be produced at lowest cost.

"The following shapes are recommended:

Sign	Shape
Railroad prewarning sign	Round
Stop sign	Octagonal
Slow signs	Diamond shape
Caution signs	Square
Speed-limit sign	Rectangular (greatest dimension vertical)
Road markers	Shaped to fit the design of marker
Guide and direction signs	Arrow or rectangular
General information	Rectangular

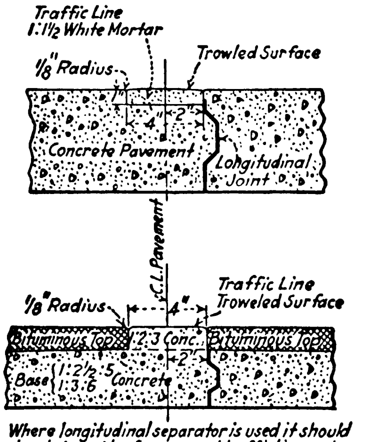
"**Sizes of Non-luminous Signs.**—While it is believed a standard size of the various signs should, in general, be used, it is also believed that in rare instances the most extreme danger points may well be marked by a larger sign than the standard in order to provide the increased warning.

"**Copy on Non-luminous Signs.**—The copy on non-luminous signs should be brief and simple and easily read. It is believed that clear English is, in general, better understood than any combination of symbols and signs and, hence, English words are recommended rather than meaningless symbols.

"Hence, it is believed that the most effective set of standard signs can be made by a proper combination of variable shapes and colors together with suitable copy and dimensions."

Reflectors.—The use of reflectors to indicate danger is effective in some locations. They have a very limited usefulness on rural roads due to difficulty in placing them where they can catch the rays of approaching headlights in time. In fog, rain, and snow storms reflectors are of small value, for their efficiency is greatly reduced.

Traffic Lane Markings.—Probably the most effective and cheapest device to increase safety is the traffic lane marking. It has a very powerful effect on drivers, and keeps them on their own side



PERMANENT TRAFFIC LINES

FIG. 252*A*.

of the road, particularly on curves and at the top of hills where the view is obscured.

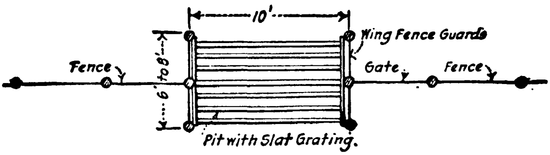
Figure 252*A* shows the method used on concrete base pavements. For the usual concrete pavement the lane is marked by the longitudinal joint line. On macadam, sheet asphalt, brick, etc. it is marked with paint or adhesive strips. On these last-named pavements where paint is used the markings are generally confined to curves and to the top of hills.

Trolley Track and Crossing Details.—Figure 40 gives the essentials of elevation, clearance, and construction details where tracks are an integral part of the pavement design. The standard shown is the New Jersey practice (see also p. 176).

Monuments—Figure 249, Miscellaneous Standards, gives good practice in the manner of permanent street monuments. The sam type of marker 6″ above ground makes a good road right-of-way marker.

Cattle Guards.—In western territory, ranch owners will often grant road right of ways for a nominal sum, but stipulate that the right of way shall not be fenced, as it would cut off part of their range from water. The boundaries of these ranges are generally fenced, and where the road passes this fence a gate must be used to prevent straying of cattle; it is more or less of a nuisance for every user of the road to open and close the gate, and generally a gap is left in the fence across which a shallow pit 2 to 3′ deep is dug. This is covered with a slat grating over which cattle will not walk but over which automobiles can be driven (Fig. 253).

Cattle Passes.—The minimum size of opening which will serve satisfactorily for tunnel passageways under highways is 6 by 6′.



Cattle guard in New Mexico.

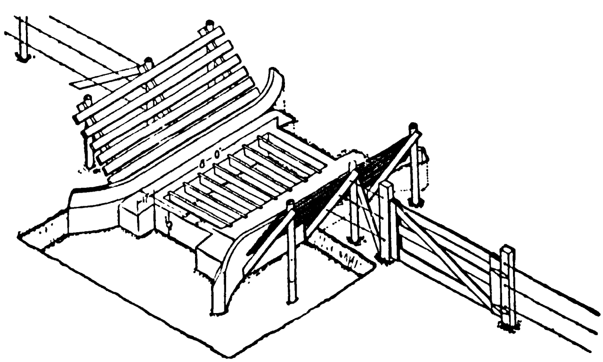


FIG. 253.—Type of crossing used extensively in Western Kansas.

Fences.—Ordinary right-of-way and snow fences are shown in Figs. 254 and 255.

Rural Highway Illumination.—Illumination is the final stage of improvement of rural highways. Effective lighting costs from \$500 to \$1,000 per mile per year. At the present stage of rural highway development (1926) it is only reasonable to incur this expense on a limited mileage of road (see Table 2, p. 6).

Illumination is a good investment on main commercial intercity hauling routes carrying a large amount of night traffic. It is also desirable and justifiable in suburban districts adjacent to cities, for it promotes residential development and increases land values.

On the ordinary improved rural road, however, it is doubtful if such a large yearly expenditure is justified.

First-class illumination increases safety of night driving and increases safe night speeds 5 to 10 miles per hour. By good illumination is meant sufficient light to nullify the glare of approaching headlights and to indicate vehicles or pedestrians by either direct illumination or silhouette effect. Under good atmospheric conditions and provided that the automobiles are operated with headlights dimmed, this can be accomplished with 200 to 250-cp. lights located approximately 20′ above the road and spaced approximately 250′ apart. This arrangement and intensity of lights, however, will not nullify the glare of full-power headlights.

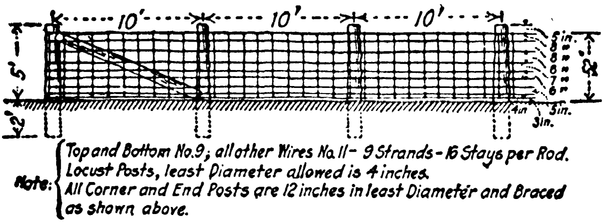


FIG. 254.—A good standard right-of-way fence.

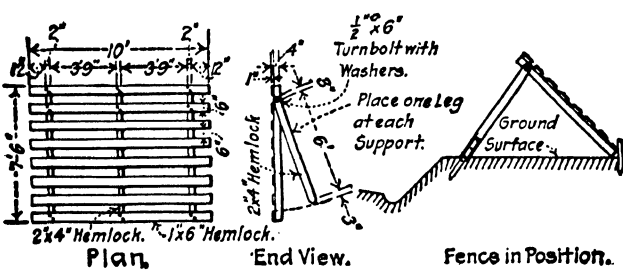


FIG. 255.—Snow fence detail.

In localities with which the author is familiar it has been found desirable to use 400-cp. lights spaced 250′ apart and located 20′ above the road. The cost of 400-cp. lights is about $40 per light per year and of 200-cp. lights about $35 per light per year (western New York 1926 cost conditions). These 400-cp. lights seem effective in nullifying objectionable glare of full-strength headlights under usual atmospheric conditions. In a bad rain storm or fog, however, any artificial illumination is a nuisance, as reflection surrounds each light with veritable curtain, which completely obscures objects beyond the light range.

THE LIGHTING OF STREETS AND HIGHWAYS

Principles and Practice Outlined in Paper Presented Nov. 13 at Annual Convention of American Society for Municipal Improvements

By Stephen Carleton Rogers

Commercial Illuminating Engineer, Street Lighting Department, General Electric Co.

"Vision is accomplished in two ways, or in a combination of two ways. Objects, persons, etc., are seen either in silhouette against the lighted background of the street surface or else by the direct light from the lamps themselves. When objects are seen in silhouette, their bulk or outline is observed; when objects are seen in perspective, or by direct light, their details or features are distinguished. All objects, etc., therefore, whether by day or by night, must be seen either in silhouette or in perspective, or by a combination of the two. These principles should be remembered, as street lighting, to a very large degree, depends upon them, no matter what class of street lighting is being considered. Vision by silhouette has been known and recognized by artists for many years, but has been appreciated by the illuminating engineer for only a comparatively short time.

"Depending upon these two methods of vision are the two main types of lighting, which may be classified as follows:

1. Large-unit lighting.
2. Small-unit lighting.

"**Large-unit Lighting.**—If a large unit equipped with a diffusing globe, for example, either a 6.6-amp. luminous (magnetite) arc lamp or a 15,000-lumen incandescent (Mazda C) lamp, be placed on the side of a street some distance away, as, for instance, 300′, objects, persons, etc., will be seen more in silhouette than in perspective. This is commonly termed "large-unit street lighting," and the illumination will be of a maximum high intensity and from a concentrated light source. It is clear, therefore, that whenever large-unit lighting is employed that silhouetting will be more generally used, particularly so when the lighting units are spaced quite a distance apart. The closer the units be spaced the more will perspective vision occur in conjunction, of course, with silhouette vision. The diffusing globe will cut down the candlepower of either of the large units just mentioned to about 960. If, for instance, twelve 80-cp. incandescent lamps be grouped together and placed in a single large globe, the resultant illumination would be practically the same as with the use of the single large unit.

"**Small-unit Lighting.**—Suppose, however, that these twelve small, 80-cp. incandescent lamps be distributed along the same street and uniformly spaced. The result then would be known as "small-unit street lighting," the illumination will be fairly uniform but of low intensity and from distributed light sources. With this type of street lighting there will not be enough intensity for either silhouette or good perspective vision. These two distinct types of street lighting also produce entirely different psychological effects, as was shown during some street-lighting tests conducted a few years ago, where it was shown that large-unit lighting is by far the superior and more essential for good street lighting from every angle.

"There is still a very important phase that should be considered, *viz.*, the status of large units versus small units, viewed from an economic point of view. A city or town naturally, since it is the party that has to pay the bills, is most vitally interested in street lighting and its costs. Since many cities have a certain appropriation for street lighting, they usually want to know what type of lighting will give the best results for that given amount of money, rather than what is the best street lighting it can get and then how much it will cost. According to prevailing conditions and rates it will cost a city approximately $100 per year for each large unit and about $25 per year for each small unit, considering these sizes just referred to. This will mean something like three large units for every twelve small units, or roughly, three times the total light flux with its accompanying increased illumination and better visual effect for the same total cost per year.

"**Street Lighting Practice.**—Street lighting, in general, may be divided into the following five classes:

"Class I. Main business streets—highly intensive or white-way illumination.

"Class II. Secondary business streets—good general illumination.

"Class III. Boulevards and main residential thoroughfares.

"Class IV. Side residential streets—comparatively low illumination.

"Class V. Interurban or main automobile highways.

"**Main Business Street Lighting.**—The main business streets of a large city may be properly separated into two subdivisions, whereas in a smaller city this distinction is lacking—it being hard to tell where one type stops and the other starts.

"*a.* Streets, whose stores are essentially retail and depend largely upon show windows for the display of their goods, merchandise, or wares—streets that are of greatest distinction.

"*b.* Important business streets largely traveled at night.

"The type of lighting that should be employed should be such as to satisfy every requirement of police protection, the motorist, the safety of the pedestrian, the merchant, the æsthetic sense, etc.

"In order to accomplish this result, the illumination of the street surface must be of high intensity so that silhouette, together with perspective vision, may be employed; cornices, facades, and other architectural details and effects must be brought out. In order to achieve these desired results, lighting units of high candle power closely spaced must be used, the units themselves must be attractive by day as well as by night, and they should be mounted upon ornamental standards, thereby doing away with unsightly overhead wires and trolley poles, etc. In large cities, two and three of these large units per post, equipped with diffusing glassware, should be specified with approximately 100′ spacing per side at 20–30′ heights. In cities of this type the buildings range from six stories and upwards, so that a light distribution with nearly equal amount of light in upper and lower hemisphere should be used. By the use of the new alabaster rippled glassware, the most pleasing effects are produced. In the small cities, where the buildings are rarely four to six stores high, excellent results may be obtained by the use of single lamp standards 15 to 18′ high, and when using the incandescent lamp the dome refractor and light alabaster rippled globe may be used, as this particular combination has been designed so as to permit the illumination of building fronts of comparatively few stories' height and at the same time considerably increase the illumination of the street surface. On the other hand, suppose that a type of distribution should be used which has practically no light in the upper hemisphere, such as would be obtained from a unit with clear globe and reflector or ordinary refractor installation (not the dome refractor and rippled globes), what would be the result? There would be a sharp shadow, a sharp cut-off on the building fronts, which would give a very unpleasant effect—absolutely wrong physiologically as well as psychologically.

"There is yet another factor that enters into lighting of this class—*viz.*, the quality of light. The chief difference between the magnetite arc lamp and the incandescent lamp of equal intensity is the color. The color of the luminous (magnetite) arc lamp is the nearest approach to daylight of any artificial illuminant commercially exploited. This white color of the magnetite arc is considered by many to be unexcelled for certain classes of lighting—especially the high intensive class where there is show-window lighting. In order to get the most effective show-window lighting a soft, warm, lighting effect is most desirable, which is obtained when the ordinary incandescent lamp is used. When the streets are lighted, the effectiveness of the show window is greatly decreased, if the color of the street lighting is the same as that of the show-window lighting, because contrast will be lacking. However, if the streets are lighted with the white color of the magnetite arc lamp, then the show windows will be in contrast with their warmer yellow light, and the attractiveness of the windows will increase and they will stand out in most beautiful fashion. This, in fact, is the accepted standard among those who are doing the highest-grade intensive lighting today.

"What would be the result if the windows should be lighted with white light (thereby revealing correct color values) and the street lighted with yellowish light? The contrast will be in the wrong direction, for the background will be cold and harsh and the foreground warm, whereas, the background must be warm and the foreground cold to satisfy. The revealing power, that is, the ability to pick up objects, etc., with white light is greater than with yellow light.

"**Types of Refractors.**—It may be pertinent here to describe some of the various types of refractors in use, and show why only the dome refractor is suitable for this class of lighting. The original refractor consisted of a series of horizontal prisms, both above and below the light source, so as to redirect all of the light flux both upward and downward at a predetermined angle in the hopes of approaching a uniform horizontal illumination. This principle is, of course, fundamentally wrong, since non-uniform illumination is of far more use in street lighting illumination. It was found that fairly good street illumination could be obtained by leaving off the prisms in the lower hemisphere, thereby not altering the downward direct light, but adding to the downward light at a given angle a large part of the upward light. In both of these refractors there is an external piece consisting of a series of vertical prisms so as to diffuse the light and lower the brilliancy, thus reducing the glare. If, now, either of these refractors is put into an ordinary diffusing globe, the refractive feature will be lost and the resultant illumination will be less than is obtained by leaving off the refractor.

"In the case of the dome refractor, however, there are no vertical prisms, two separate series of horizontal prisms in the upper hemisphere being used to redirect the upward light, thereby utilizing a larger spherical angle and a consequently greater amount of light flux. In order to reduce glare, a rippled glass globe is used which, when very lightly coated with alabaster, does not entirely nullify the directive feature of the refractor and at the same time spreads sufficient light upward for fairly high buildings. The asymmetric type of refractor, of course, takes even more light from the building fronts and should be used only for other classes of streets as described later.

"**Lighting of Secondary Business Streets.**—This classification should include those business streets not coming under Class I, in other words, those having comparatively little travel at night, those including largely wholesale stores, warehouses, factories, etc. In streets of this class the same type of illuminant should be used, the same intensity of unit, but longer spacing between lamps than in Class I. This is more advisable than going to the next smaller size unit and retaining the same spacing, although the cost per year may be the same, better visual effects may be obtained by fewer larger units than by more smaller units. The lighting of these streets is mostly done for police and fire protection.

"**Boulevards and Main Residential Thoroughfares Lighting.**—In this classification should also be included park lighting. These streets are used by automobilists and pedestrians, and the needs of each should be satisfied. Lamps best suited for this type of lighting should be of the ornamental types used for Classes I and II lighting, but spaced farther apart. Of course, the best results will be obtained by using the large 6.6-amp. luminous arc lamps or the 15,000-lumen incandescent lamps—however, very fine results can be obtained with the 4- or 5-amp. luminous arc lamps or the 6000- or 10,000-lumen incandescent lamps with rippled globes. It might be added that a semiornamental bracket type of unit giving the same light distribution can be used where overhead wiring is available, thereby saving the additional expense of ornamental standards and underground wiring.

"**Lighting Side Residential Streets.**—On account of the small amount of motor travel on this type of street, a less expensive unit can be well utilized here, one employing a 4000-lumen incandescent lamp serving the purpose admirably. A novel as well as utilitarian unit has recently been designed for this work. It consists of a single piece of glazed porcelain which acts as insulator and prismatic glass holder as well.

"**Interurban or Main Automobile Highway Lighting.**—These roads are the main traffic routes connecting towns and cities. The problem of lighting these is entirely changed, due to the ever-increasing auto traffic and headlight glare. Another factor enters into this class of lighting especially, namely, the character of the road surface. There are for all practical purposes three main types of road surfaces:

"1. Non-reflecting, non-diffusing—*viz.*, country roads with possibly a light coating of oil to lay the dust.

"2. Diffusing—light-dust, reinforced-concrete, cement roads.

"3. Reflecting—bituminous-macadam, asphalt, wood-block, etc., roads.

"The majority of these highways are of the latter two types, with the reflecting types in greater abundance. Because of these improved roads, it has been possible to design and utilize a special unit, consisting of a number of nested parabolic reflectors, making it possible for one lamp to be in the focus of six reflectors and thereby distributing the light along the road without any spilled or wasted light, as is the case with other than asymmetric distribution of light. In the case of the black-surface roads, it is possible, using 2500-lumen (250-cp.) lamps with spacing of 300′ and height of 30′, to make the road surface a veritable ribbon of light—to use the road surface as a secondary light source—to reduce glare from approaching automobile headlights by decreasing the contrast between them and their background. This type of lighting is one of the greatest advances in street illumination that has been made—it is very economical, especially when it is considered that it costs approximately $30,000 to $40,000 per mile for investment of the road itself as against $3000 per mile for the lighting units and lines, etc."

maintenance of the roads costs about $5000 per mile, as against $1000 per mile for the lighting. Some of the obvious advantages of this kind of lighting are as follows;

"I. Prevents accidents:
"1. By showing up dangerous curves.
"2. By reducing headlight glare.
"3. By illuminating signs, sides of roads, and obstacles.
"II. Adds to comfort of night driving:
"1. By relieving eyestrain.
"2. By assisting in making repairs.
"3. By discouraging holdups.
"III. Increases night traffic and thereby relieves day congestion.
"IV. Decreases running time and increases road capacity.
"V. Helps to bring electricity to the farm by providing a pole line.
"VI. Increases real estate values:
"1. By tending to extend the city along highways.
"2. By bringing electrical conveniences.

"As an illustration of what might be accomplished in the way of highway lighting from an economic point of view, take, for example, Massachusetts (and the same might apply to any other state). There are at present, roughly, 1500 miles of improved state highway, which should be sceintifisally lighted. This would mean an initial investment of about $5,000,000, with a yearly maintenance cost of $1,500,000 (figuring an average cost of $60 per lamp year to the city or state). A few years ago drastic headlight laws were enacted, which, it has been conservatively estimated, cost the automobile owners more than $2,500,000 at that time, plus the additional amount necessary for their upkeep. If this money had been expended in the proper direction, headlights would be unnecessary and safety to all would have been increased.

Really complete lighting similar to city streets undoubtedly increases the amount of traffic a road can safely carry at night, and for main commercial roads, where the freighting is carried on night and day, good illumination costing about $800 to $1000 per mile per year is undoubtedly more than justified on the score of being the cheapest and safest means of increasing road-carrying capacity at night.

CHAPTER XI

MATERIALS AND TESTS

Introductory Note.—This chapter duplicates a small amount of the data from different chapters but has not been rearranged, as such duplications add to the ease of reference.

Materials.—The selection of materials is an important part of the design. Most municipal and state departments have well-equipped laboratories for testing stone, gravels, brick, bitumens, cements, etc. The object of these tests is to determine the physical and chemical properties that have a particular bearing on the action of the materials under construction conditions. While these conditions are not attained, they are approximated, and by a comparison of the laboratory results with the actual performance of the different materials in practice a relation can be established that is useful as a basis for judgment.

The authors are greatly indebted to H. S. Mattimore and J. E. Myers, who have rearranged and brought up to date much of the material on tests and their significance.

This chapter gives a brief statement of the desirable qualities and tests for:

1. Top course, macadam stone.
2. Screenings.
3. Bottom course, macadam stone.
4. Bottom course and subbase fillers.
5. Brick.
6. Bituminous binders.
7. Cement (Portland).
8. Fine aggregate for cement or bituminous concretes.
9. Coarse aggregate for cement concrete.
10. Sand-clay and gravel tests.

Instructions for sampling materials are given in Chap. XII, page 752.

1. STONE FOR THE SURFACING OF MACADAM ROADS

Stone for use in the surfacing of a macadam road should be hard and tough to withstand the abrasive action of team traffic and the vibratory action of high-speed motor vehicles, and should not contain any minerals that are likely to disintegrate rapidly under the influence of weather conditions.

To determine the relative hardness, toughness, and power to resist abrasive and impact action of traffic, stones are subjected to the following tests:[1]

[1] American Society of Testing Materials.

1. Abrasion.
2. Hardness.
3. Toughness.
4. Specific gravity.
5. Absorption.
6. Fracture.
7. Geological classification.

"**Abrasion Test.**[1]—The machine shall consist of one or more hollow iron cylinders; closed at one end and furnished with a tightly fitting iron cover at the other; the cylinders to be 20 cm. in diameter and 34 cm. in depth, inside. These cylinders are to be mounted on a shaft at an angle of 30° with the axis of rotation of the shaft.

"At least 30 lb. of coarsely broken stone shall be available for a test. The rock to be tested shall be broken in pieces as nearly uniform in size as possible, and as nearly 50 pieces as possible shall constitute a test sample. The total weight of rock in a test shall be within 10 g. of 5 kg.

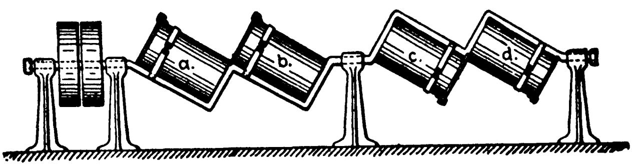


FIG. 256.—Deval rattler.

"All test pieces shall be washed and thoroughly dried before weighing. Ten thousand revolutions, at the rate of between 30 and 33 per minute, shall constitute a test. Only the percentage of materials worn off which will pass through a 0.16-cm. ($\frac{1}{16}''$) mesh sieve shall be considered in determining the amount of wear. This may be expressed either as the percentage of the 5 kg. used in the test, or the French coefficient, which is in more general use, may be given; that is,

Coefficient of wear $= 20 \times \frac{20}{w} = \frac{400}{w}$, where w is the weight in grams of the detritus under 0.16 cm. ($\frac{1}{16}''$) in size per kilogram of rock used.

TABLE 118.—CONVERSION TABLE PER CENT OF WEAR TO FRENCH COEFFICIENT

French coefficient	% of wear	French coefficient	% of wear
20.0	2	8.0	5
13.3	3	6.7	6
10.0	4	5.7	7

"**Hardness.**[1]—Hardness is determined by a Dorry machine. A stone cylinder 25 cm. in diameter, obtained by a diamond core drill from the material to be tested, is weighed and placed in the machine so that one end rests on a horizontal cast-iron grinding disc with a pressure of 25 g. per square centimeter. The disk is revolved 1000 times, during which standard crushed quartz sand about $1\frac{1}{2}$ mm. in diameter is automatically fed to it. The cylinder is then removed and weighed and the coefficient of hardness obtained by the formula $20 - \frac{1}{3}$ the loss in weight, expressed in grams. In order to get reliable results, two cylinders are generally used, each one being reversed end for end during the test.

[1] American Society of Testing Materials.

"**Test for Toughness.**[1]—1. Test pieces may be either cylinders or cubes, 25 mm. in diameter and 25 mm. in height, cut perpendicular to the cleavage of the rock. Cylinders are recommended as they are cheaper and more easily made.

"2. The testing machine shall consist of an anvil, 50 kg. in weight, placed on a concrete foundation. The hammer shall be of 2-kg. weight, and dropped upon an intervening plunger of 1-kg. weight, which rests on the test piece. The lower or bear surface of this plunger shall be of spherical shape having a radius of 1 cm. This plunger shall be made of hardened steel, and pressed firmly upon the test piece by suitable springs. The test piece shall be adjusted so that the center of its upper surface is tangent to the spherical end of the plunger.

"3. The test shall consist of a 1-cm. fall of the hammer for the first blow, and an increased fall of 1 cm. for each succeeding blow until failure of the test piece occurs. The number of blows necessary to destroy the test piece is used to represent the toughness, or the centimeter-grams of energy applied may be used.

"**Determination of the Apparent Specific Gravity of Rock.**[1]—The apparent specific gravity of rock shall be determined by the following method: First, a sample weighing between 29 and 31 g. and approximately cubical in shape shall be dried in a closed oven for 1 hr. at a temperature of 100°C. (230°F.) and then cooled in a desiccator for 1 hr.; second, the sample shall be rapidly weighed in air; third, trial weighings in air and in water of another sample of approximately the same size shall be made in order to determine the approximate loss in weight on immersion; fourth, after the balances shall have been set at the calculated weight, the first sample shall be weighed as quickly as practicable in distilled water having a temperature of 25°C. (77°F.); fifth, the apparent specific gravity of the sample shall be calculated by the following formula:

$$\text{Apparent specific gravity} = \frac{W}{W - W_1},$$

in which W = weight in grams of the sample in air and W_1 = the weight in grams of the sample in water just after immersion.

The apparent specific gravity shall be the average of three determinations made on three different samples.

"**Determination of the Absorption of Water per Cubic Foot of Rock.**[1]—The absorption of water per cubic foot of rock shall be determined by the following method: First, a sample weighing between 29 and 31 g. and approximately cubical in shape shall be dried in a closed oven for 1 hr. at a temperature of 110°C. (230°F.) and then cooled in a desiccator for 1 hr.; second, the sample shall be rapidly weighed in air; third, trial weighings in air and in water of another sample of approximately the same size shall be made in order to determine the approximate loss in weight on immersion; fourth, after the balances shall have been set at the calculated weight, the first sample shall be weighed as quickly as possible in distilled water having a temperature of 25°C. (77°F.); fifth, allow the sample to remain 48 hrs. in distilled water maintained as nearly as practicable at 25°C. (77°F.), at the termination of which time bring the water to exactly this temperature and weigh the sample while immersed in it; sixth, the number of pounds of water absorbed per cubic foot of the sample shall be calculated by the following formula:

$$\text{Pounds of water absorbed per cubic foot} = \frac{W_2 - W_1}{W - W_1} \times 62.24, \text{ in which}$$

W = the weight in grams of sample in air, W_1 = the weight in grams of sample in water just after immersion, W_2 equals weight in grams in water after 48 hours immersion and 62.24 = the weight in pounds of a cubic foot of distilled water having a temperature of 25°C. (77°F.).

"Finally, the absorption of water per cubic foot of the rock, in pounds, shall be the average of three determinations made on three different samples according to the method above described.

"**Fracture.**—Stone suitable for road work should crush in cubical shapes rather than in thin, flat pieces, and preferably with rough jagged fracture that it may interlock firmly under action of the roller.

"**Geological Classification.**—The geological classification is determined from an examination with a microscope or powerful hand glass, and a consideration of its origin. Great refinements are avoided as the general classification is all that is necessary to the highway engineer after the physical qualities are ascertained by test."

[1] American Society of Testing Materials.

TABLE 119.—GEOLOGICAL CLASSIFICATION

Class	Type	Family
I. Igneous	1. Intrusive (plutonic)	a. Granite b. Syenite c. Diorite d. Gabbro e. Peridotite
	2. Extrusive (volcanic)	a. Rhyolite b. Trachyte c. Andesite d. Basalt and diabase
II. Sedimentary	1. Calcareous	a. Limestone b. Dolomite
	2. Siliceous	a. Shale b. Sandstone c. Chert (flint)
III. Metamorphic	1. Foliated	a. Gneiss b. Schist c. Amphibolite
	2. Non-foliated	a. Slate b. Quartzite c. Eclogite d. Marble

TABLE 120.—TAKEN FROM BULLETIN 31, U. S. OFFICE OF PUBLIC ROADS

Rock varieties	Per cent wear[a]	Toughness	Hardness	Cementing value	Specific gravity
Granite	3.5	15	18.1	20	2.65
Biotite-granite	4.4	10	16.8	17	2.64
Hornblende-granite	2.6	21	18.3	30	2.76
Augite-syenite	2.6	10	18.4	24	2.80
Diorite	2.9	21	18.1	41	2.90
Augite-diorite	2.8	19	17.7	55	2.98
Gabbro	2.8	16	17.9	29	3.00
Peridotite	4.0	12	15.2	28	3.40
Rhyolite	3.7	20	17.8	48	2.60
Andesite	4.7	11	13.7	189	2.50
Fresh basalt	3.3	23	17.1	111	2.90
Altered basalt	5.3	17	15.6	239	2.75
Fresh diabase	2.0	30	18.2	49	3.00
Altered diabase	2.5	24	17.5	156	2.95
Limestone	5.6	10	12.7	60	2.70
Dolomite	5.7	10	14.8	42	2.70
Sandstone	6.9	26	17.4	90	2.55
Feldspathic sandstone	3.3	17	15.3	119	2.70
Calcareous sandstone	7.4	15	8.3	60	2.66
Chert	10.8	15	19.4	27	2.50
Granite-gneiss	3.8	12	17.7	26	2.68
Hornblende-gneiss	3.7	10	17.1	30	3.02
Biotite-gneiss	3.2	19	17.5	41	2.76
Mica-schist	4.4	10	17.8	30	2.80
Biotite-schist	4.0	—	—	16	2.70
Chlorite-schist	4.2	—	—	24	2.90
Hornblende-schist	3.7	21	16.5	53	3.00
Amphibolite	2.9	10	19.0	29	3.00
Slate	4.7	12	11.5	102	2.80
Quartzite	2.9	19	18.4	17	2.70
Feldspathic quartzite	3.2	17	18.3	21	2.70
Pyroxene quartzite	2.3	27	18.6	17	3.00
Eclogite	2.4	31	17.4	21	3.30
Epodosite	3.6	16	16.0	47	3.03

[a] To convert % of wear to French coefficient, see table on page 706.

TABLE 121.—FROM ANNUAL REPORT N. Y. STATE HIGHWAY COMMISSION, 1914

County	Number of complete tests	Number of partial tests (no core piece)	Weight lbs. per cu. ft.	Water absorbed, lbs. per cu. ft.	French coefficient of abrasion	Hardness	Toughness	Weighted value
Calcareous Sandstone								
Erie	5	...	167	0.65	9.5	12.9	13.4	68
Saratoga	6	...	169	0.31	10.1	15.9	13.8	76
Steuben	4	1	162	1.44	9.4	15.1	13.1	72
Dolomite								
Clinton	6	...	175	0.41	11.9	15.8	12.7	80
Dutchess	4	1	174	0.43	12.4	17.3	11.0	84
Essex	4	...	173	0.42	13.5	16.9	15.8	90
Franklin	4	...	174	0.51	9.5	14.9	12.1	70
Fulton	4	...	176	0.15	11.8	16.1	14.4	82
Herkimer	17	...	173	0.67	8.4	13.1	6.7	58
Monroe	13	2	171	1.07	10.3	14.8	8.2	69
Montgomery	8	...	174	0.39	10.6	14.7	11.3	73
Niagara	11	...	168	1.50	6.5	14.0	7.0	55
Saratoga	8	...	174	0.33	8.6	15.5	9.2	66
St. Lawrence	31	...	174	0.65	10.5	15.7	9.9	73
Washington	6	...	175	0.29	10.7	15.1	10.5	73
Dolomitic Limestone								
Dutchess	8	1	176	0.46	9.0	14.9	10.9	68
Herkimer	4	1	170	0.47	11.3	16.7	8.2	76
Montgomery	8	1	175	0.41	13.0	15.8	12.4	83
Niagara	7	...	166	2.19	9.5	13.1	7.8	63
St. Lawrence	7	...	168	0.38	9.2	16.8	6.8	68
Washington	4	...	175	0.36	13.7	16.1	10.8	84
Wayne	4	...	173	0.59	10.2	15.5	8.7	71
Gabbro								
Essex	46	1	176	0.29	7.6	17.3	6.9	64
Warren	4	...	183	0.37	10.1	17.7	9.8	75
Gneiss								
Clinton	5	...	185	0.27	10.5	17.2	11.3	78
Dutchess	8	1	172	0.58	7.0	17.1	9.1	64
Essex	29	2	176	0.31	8.4	17.1	8.1	68
Franklin	8	...	178	0.50	6.2	16.1	7.8	59
Fulton	12	1	169	0.25	11.1	17.8	11.5	80
Hamilton	11	...	173	0.37	8.2	17.0	5.8	64
Jefferson	26	1	171	0.23	11.1	17.3	12.0	80
Lewis	6	...	167	0.27	9.6	17.9	10.6	75
Orange	7	...	179	0.38	7.1	17.1	6.4	62
Putnam	10	1	172	0.32	8.5	16.6	7.5	66
Saratoga	7	...	180	0.20	10.0	17.0	8.5	72
St. Lawrence	52	...	172	0.27	9.7	17.5	10.2	74
Warren	30	2	173	0.30	7.5	17.3	6.5	64
Washington	4	...	170	0.29	8.5	17.1	10.9	71
Westchester	37	2	171	0.39	8.3	16.9	7.8	67
Granite								
Essex	5	...	171	0.38	7.5	18.0	5.1	64
Franklin	6	...	165	0.31	8.7	17.9	9.4	71
Hamilton	5	...	165	0.36	9.9	18.1	9.0	75
Jefferson	23	1	166	0.23	12.1	18.4	10.1	83
Lewis	8	...	166	0.36	10.9	18.4	9.2	79
Oneida	6	...	166	0.13	10.2	18.9	8.2	77
St. Lawrence	30	...	165	0.25	9.9	18.3	8.1	74
Warren	5	...	165	0.45	7.9	17.9	7.7	67

TABLE 121—*Continued*

County	Number of complete tests	Number of partial tests (no core piece)	Weight, lbs. per cu. ft.	Water absorbed, lbs. per cu. ft.	French coefficient of abrasion	Hardness	Toughness	Weighted value
Limestone								
Albany	13	7	168	0.60	7.9	14.3	6.4	59
Cayuga	34	6	170	0.49	8.8	14.9	7.8	64
Clinton	14	2	170	0.28	8.2	14.1	5.3	58
Columbia	12	...	170	0.28	9.1	15.3	9.2	67
Erie	9	3	167	0.57	8.1	16.6	8.3	66
Fulton	6	1	168	0.21	7.7	15.5	6.5	60
Genesee	6	3	169	0.26	8.0	15.0	8.2	62
Greene	11	...	169	0.36	11.1	16.4	8.9	75
Herkimer	17	9	169	0.26	8.7	14.8	8.2	64
Jefferson	105	44	169	0.28	7.6	15.1	6.4	59
Lewis	26	20	169	0.32	6.9	14.1	6.2	55
Madison	16	1	169	0.23	8.4	14.7	7.7	62
Monroe	4	...	168	0.27	8.1	14.1	7.4	60
Montgomery	12	2	169	0.24	8.5	15.3	8.0	64
Niagara	11	1	168	0.84	7.1	12.8	6.5	53
Oneida	31	19	169	0.29	7.8	13.8	6.6	58
Onondaga	25	1	170	0.38	8.9	15.7	8.4	67
Ontario	11	...	169	0.39	10.2	15.9	10.2	73
Otsego	7	2	169	0.32	8.1	14.1	6.3	59
Rensselaer	4	1	171	0.21	7.5	15.0	5.3	58
Saratoga	5	...	170	0.24	8.7	13.7	7.0	60
Schoharie	29	2	169	0.34	8.1	14.9	6.7	61
Seneca	7	3	169	0.21	9.4	15.3	7.9	67
Ulster	12	3	170	0.25	8.1	15.6	7.4	63
Warren	5	...	170	0.24	8.9	15.7	7.4	66
Washington	5	3	169	0.34	7.9	15.5	6.9	62
Marble								
Dutchess	4	...	178	0.30	7.3	14.2	6.0	56
Quartzite								
Columbia	16	...	168	0.28	16.5	18.3	17.1	103
Dutchess	8	2	166	0.36	13.5	18.8	11.8	90
Rensselaer	10	...	166	0.49	12.1	18.7	14.8	89
Washington	12	..	167	0.40	14.6	18.9	16.3	98
Sandstone								
Allegheny	8	..	156	2.10	8.4	13.4	9.1	61
Broome	11	...	165	1.29	7.8	12.9	10.5	60
Cayuga	4	1	167	1.16	7.8	12.1	10.5	58
Chenango	15	1	164	1.58	8.7	11.2	10.4	59
Clinton	14	...	163	0.71	11.7	18.5	11.0	83
Delaware	53	2	167	1.45	7.0	12.7	8.5	55
Erie	8	1	159	2.10	6.3	5.1	7.8	37
Franklin	5	...	157	1.06	9.7	17.9	7.1	72
Greene	6	...	169	0.62	8.6	14.5	8.1	63
Herkimer	4	..	160	2.50	10.9	16.4	10.7	76
Jefferson	8	...	156	1.46	8.3	16.2	6.3	64
Livingston	4	...	160	3.02	8.8	9.6	8.8	54
Madison	5	...	163	2.15	9.9	13.9	8.6	66
Niagara	7	...	158	1.78	9.0	16.4	8.2	68
Orleans	8	...	155	2.18	11.8	14.4	8.1	72
Otsego	21	1	162	1.75	8.4	11.9	9.6	59
Saratoga	5	...	163	0.36	10.7	18.0	8.7	77

TABLE 121—*Continued*

County	Number of complete tests	Number of partial tests (no core piece)	Weight, lbs. per cu. ft.	Water absorbed, lbs. per cu. ft.	French coefficient of abrasion	Hardness	Toughness	Weighted value
SANDSTONE.—*Continued*								
Schoharie...	6	3	165	1.21	9.4	15.2	11.7	70
Schuyler....	4	...	162	2.14	8.1	11.6	10.6	58
Seneca......	5	...	165	0.86	11.0	13.9	15.8	77
Steuben.....	22	3	157	2.79	8.3	9.3	10.0	54
St. Lawrence	16	...	159	0.79	10.0	17.8	7.2	73
Sullivan....	30	4	164	1.26	6.5	14.9	8.2	58
Ulster......	8	...	166	0.64	8.0	14.3	8.1	61
Wyoming...	7	...	159	2.54	6.0	5.1	7.9	36
SANDY GRIT								
Albany.....	5	...	167	0.75	7.5	13.2	7.2	56
Columbia...	12	...	168	0.32	10.7	15.9	11.7	76
Dutchess....	10	2	168	0.57	8.1	16.2	11.5	68
Greene.....	13	...	169	0.48	7.1	15.6	9.5	62
Montgomery	4	...	166	1.39	10.1	11.3	11.8	65
Rensselaer..	10	...	169	0.44	9.1	15.9	9.4	69
Saratoga....	5	...	168	0.99	11.8	15.2	11.9	78
Schenectady	4	...	165	1.10	9.2	14.6	9.5	66
Ulster......	7	...	169	0.59	7.5	13.8	10.2	60
SYENITE								
Essex.......	7	...	184	0.52	7.7	17.1	6.7	64
Franklin....	4	...	171	0.45	10.1	18.3	8.0	75
Herkimer...	13	...	174	0.16	12.5	18.0	11.6	85
Jefferson....	7	...	176	0.34	12.4	18.1	14.5	88
TRAP								
Rockland...	12	...	183	0.39	13.2	17.6	1.64	91

The following quotation from *Bulletin* 31 Office of Public Roads describes the characteristics of the three groups:

"**Igneous Rocks.**—All rocks of the igneous class are presumed to have solidified from a molten state, either upon reaching the earth's surface or at varying depths beneath it. The physical conditions, such as heat and pressure, under which the molten rock magma consolidated, as well as its chemical composition and the presence of included vapors, are the chief features influencing the structure. Thus, we find the deep-seated, plutonic rocks coarsely crystalline with mineral constituents well defined, as in case of granite rocks, indicating a single, prolonged period of development, whereas the members of the extrusive or volcanic types, solidifying more rapidly at the surface, are either fine grained or frequently glassy and vesicular, or show a porphyritic structure. This structure is produced by the development of large crystals, generally by a recurrence of mineral growth during the effusive period of magmatic consolidation.

"In the arrangement of the rock families from a mineralogical standpoint, it will be noted that the plutonic-rock types, granite, syenite, and diorite, are represented by their equivalent extrusive varieties, rhyolite and andesite, and that diabase has been included, somewhat arbitrarily, with basalt, as a volcanic representative of gabbro. These latter rocks are of special interest, owing to their wide distribution and general use in road construction. They occur in the forms of dykes, intruded sheets, or volcanic flows, and vary in structure from glassy prophyritic (typical basalt) to wholly crystalline and even granular (diabase). Their desirable qualities for road building are caused to a large extent by a peculiar interlocking of the mineral components (ophitic structure), yielding a very tough and resistant material well qualified to sustain the wear of traffic.

"Igneous rocks vary in color from the light gray, pink, and brown to the acid granites, syenites, and their volcanic equivalents (rhyolite, andesite, etc.) to the dark steel gray or black of the basic gabbro, peridotite, diabase, and basalt. The darker varieties are commonly called "trap." This term is in very general use and is derived from *trappa*, Swedish for stair, because rocks of this kind on cooling frequently break in large tabular masses, as may be seen in the exposures of diabase on the west shore of the Hudson River from Jersey City to Haverstraw.

"**Sedimentary Rocks.**—The sedimentary rocks as a class represent the consolidated products of former rock disintegration, as in the case of sandstone, conglomerate, shale, etc., or they have been formed from an accumulation of organic remains chiefly of a calcareous nature, as is true of limestone and dolomite. These fragmental or clastic materials have been transported by water and deposited mechanically in layers on the sea or lake bottoms, producing a very characteristic bedded or stratified structure in many of the resulting rocks.

"In the case of certain oolitic and travertine limestones, hydrated iron oxides, siliceous deposits, such as geyserite, opal, flint, chert, etc., the materials have been formed chiefly by chemical precipitation and show generally a concentric or colloidal structure.[1] Oölitic and pisolitic limestones consist of rounded pea-like grains of calcic carbonate held together by a calcareous cement. Travertine is the so-called "onyx marble" of Mexico and Arizona. It is a compact rock, concentric in structure, and formed by the precipitation of carbonate of lime from the waters of springs and streams.

[1] MERRILL, G. P., "Rocks, Rock Weathering, and Soils," pp. 104–114, 1897.

"Loose or unconsolidated rock débris of a prevailing siliceous nature comprise the sands, gravels, finer silts, and clays (laterite, adobe, loess, etc.). Shell sands and marls, on the other hand, are mainly calcareous, and are formed by an accumulation of the marine shells and of lime-secreting animals. Closely associated with the latter deposits in point of origin are the beds of diatomaceous or infusorial earth composed almost entirely of the siliceous casts of diatoms, a low order of seaweed or algæ.

"This unconsolidated material may pass by imperceptible gradations into representative rock types through simple processes of induration. Thus clay becomes shale, and that in turn slate, without necessarily changing the chemical or mineralogical composition of the original substance.

"Such terms as flagstone, freestone, brownstone, bluestone, graystone, etc., are generally given to sandstones of various colors and composition, while puddingstone, conglomerate, breccia, etc., apply to consolidated gravels and coarse feldspathic sands.

"The calcareous rocks are of many colors, according to the amount and character of the impurities present.

"**Metamorphic Rocks.**—Rocks of this class are such as have been produced by prolonged action of physical and chemical forces (heat, pressure, moisture, etc.) on both sedimentary and igneous rocks alike. The foliated types (gneiss, schist, etc.) represent an advanced stage of metamorphism on a large scale (regional metamorphism), and the peculiar schistose or foliated structure is due to the more or less parallel arrangement of their mineral components. The non-foliated types (quartzite, marble, slate, etc.) have resulted from the alteration of sedimentary rocks without materially affecting the structure and chemical composition of the original material.

"Rocks formed by contact metamorphism and hydration, such as hornfels pyroxene marble, serpentine, serpentineous limestone, etc., are of great interest from a petographical standpoint, but are rarely of importance as road materials.

"The color of metamorphic rock varies between gray and white of the purer marbles and quartzites to dark-grey and green of the gneiss, schists, and amphibolites. The green varieties are commonly known as greenstones, or greenstone schists."

Interpretation of Tests.—It has been found impractical to specify definite qualities of stone for use in macadam highways. Economy and practical engineering demand that all available sources be considered. Tests are made to determine the relative qualities of stone from these different sources and the results used as a guide for selection.

In the work of the New York State Highway Commission all tests are tabulated geographically, using a county as a unit. Table 121 is compiled from the records of this department. It will be noted that comparisons are made in different classifications only, as it is considered that conclusions should not be drawn from a comparison of tests procured from materials having different origins and composed of different minerals.

For the purpose of ready comparison a figure known as the "weighted value" has been introduced (see last column of table). This is computed by giving relative weights of 3 to the French coefficient, 2 to the hardness, 1 to the toughness values, and adding three together. These relative weights were determined from a consideration of the amount of material used in the different tests and the personal equation of running them.

By consulting these tables, the available rocks of different classifications in various sections throughout New York State can be determined readily, and as new tests are completed they are compared with good average material from that section.

Conclusions.—Trap (diabase), granite, gneiss, quartzite, sandstone, and limestone are the most common rocks and when found in a good state of preservation make good surfacing materials (see Chap. VI, pp. 440 and 446, for acceptable test values, macadam top course stones).

As generally found, trap is uniform in hardness and toughness, making an excellent material for use in top course.

Granite and gneiss, where they occur with hornblende replacing a large percentage of the quartz, make an excellent surfacing stone.

Quartzites when found in a good state of preservation are hard and tough. They should not be confused with crystalline quartz, which is hard but brittle.

Sandstones are extremely variable and only the better varieties should be used.

Limestones range from the fine-grained dense products which are hard and tough to the coarse-grained soft products which are not suitable for surfacing.

2. SCREENINGS

Screenings act as a filler and binder for water-bound macadam and as a partial filler for bituminous macadam. For use in water-bound construction the main mineral constituent is the most essential feature to be considered, as this must be a material that will form a binder and "puddle" readily when subjected to the action of a road roller and water.

Limestone screenings have proved the most efficient as a binder in water-bound construction, although trap and some other igneous rocks can be bound with their own dust by repeated puddling. Screenings consisting mainly of quartz have not been used successfully in water-bound construction except by the addition of some limestone screenings. The use of a percentage of clay or loam as a binder is not advisable except where the cost of limestone screenings would be prohibitive.

Laboratory methods for testing the cementing power of rock powders are available, but the results obtained are erratic and undependable.

In plain water-bound roads it is often necessary to mix some limestone screenings, fine sandy loam, or even a small percentage of clay loam with trap, granite, sandstone, quartzite, or gneiss screenings to get a good bond and prevent raveling in dry weather.

3. BOTTOM-COURSE MACADAM STONE

As the bottom stone simply spreads the wheel loads transmitted through the top course and is not directly subjected to the traffic action, almost any stone that breaks into cubical irregular shapes that will not air or water slake, and that is hard enough to stand the action of the roller during construction, will be satisfactory.

Any of the materials listed in Table 120, except shale and slate, can be used, provided they are not rotten from long exposure in the air. The different available varieties are usually tested in the same manner as for top stone in order to pick the best. Blast-furnace crushed slag makes an excellent bottom course but is not uniform enough for top course.

4. FILLERS

Fillers are used in the bottom course to fill the voids between the crushed stone and to prevent rocking or sidewise movement of the larger pieces.

They should be easy to manipulate in placing, should not soften when wet, or draw water up from the subgrade by capillary action.

The materials most used for macadam bottom are:

Coarse sandy loam.
Coarse sand.
Gravel with large excess of fine material.
Stone screenings.

The fitness of the material can be determined by inspection and by wetting a handful; if it gets sticky or works into a soft mud it should not be used.

Boulder or Telford Base Fillers.—This type of construction requires the use of clean, hard, gravel fairly coarse (at least 40% retained on 1/4" screen) with not over 15% of loam or silt content, or broken stone, slag, or chert. If broken stone is used, the voids of the stone must be filled with the same class of sand or screenings as stipulated for macadam bottom.

5. VITRIFIED BRICK

Bricks must withstand the same destructive agencies as described for top stone. They must be uniform in size, tough, hard, dense, evenly burned, and, on account of their peculiar shape, must have a high resistance against rupture. These properties are tested by the standard methods adopted by the American Brick Manufacturers' Association described below.

It should be understood that bricks suitable for paving are manufactured in a different way and of different materials than ordinary building bricks.

"**The materials for molding any paving brick, must be of a peculiar character which will not melt and flow when exposed to an intense heat for a number of days but will gradually fuse and form vitreous combinations throughout while still retaining its form.**

"The resulting brick must be a uniform block of dense texture in which the original stratification and granulation of the clay has been wholly lost by fusion which has stopped just short of melting the clay and forming glass.

"The clay while fusing must shrink equally throughout, thus causing the brick to be without laminations or of any exterior vitrified crust differing from the interior."[1]

The great majority of paving bricks are made in Ohio, Illinois, Indiana, Pennsylvania, West Virginia, and New York. They are classed as shale or fire-clay brick.

BRICK

"Paving brick shall be reasonably perfect in shape—shall be free from marked warping or distortion, and shall be uniform in size, so as to fit closely together and to make a smooth pavement. All brick shall be homogeneous in texture and free from laminations and seams. All brick shall be evenly burned and thoroughly vitrified.

"Soft, brittle, cracked, or spalled brick, or brick kiln marked to a height or depth of over 3/64" will be rejected.

"If brick have rounded corners, the radius shall not be greater than 1/16"

"Brick must not have less than two nor more than four vertical lugs or projections not more than 1/2" wide, on one side of each brick, the total area of all lugs being not more than 3 sq. in., so that when laid there shall be a separation between the bricks of at least 1/8" and not more than 1/4". The imprint, or name of the brick, or maker, if used, shall not be by means of recessed, nor by raised letters. The two ends of the brick shall have a semicircular groove, with a radius of not less than 1/8" and not more than 1/4". Grooves shall be so located that when the bricks are laid together the grooves shall match perfectly; grooves shall be horizontal when brick is laid in pavement.

"All brick shall not be less than 3 1/4 by 3 3/4 by 8 1/2" nor more than 3 1/2 by 4 by 9" in size.

"All brick shall be subject to tests for abrasion and impact, for absorption according to the standard methods prescribed by the National Brick Manufactures' Association, as follows:

"**The Rattler.**—The machine shall be of good mechanical construction, self-contained, and shall conform to the following details of material and dimensions, and shall consist of barrel, frame, and driving mechanism as herein described.

"**The Barrel.**—The barrel of the machine shall be made up of the heads, headliners, and staves.

"The heads shall be cast with trunnions in one piece. The trunnion bearings shall not be less than 2 1/2" in diameter or less than 6" in length.

"The heads shall not be less than 3/4" thick nor more than 7/8". In outline they shall be a regular 14-sided polygon inscribed in a circle 28 3/8" in diameter. The heads shall be provided with flanges not less than 3/4" thick and extending outward 2 1/2" from the inside face of head to afford a means of fastening the staves. The flanges shall be slotted on the outer edge, so as to provide for two 3/4" bolts at each end of each stave, said slots to be 13/16" wide and 2 3/4" center to center. Under each section of the flanges there shall be a brace 3/8" thick and extending down the outside of the head not less than 2". Each slot shall be provided with recess for bolt head, which shall act to prevent the turning of same. There shall be for each head a cast-iron headliner 1" in thickness and conforming to the outline of the head, but inscribed in a circle 28 1/8" in diameter. This liner or wear plate shall be fastened to the head by seven 5/8" cap screws, through the head from the outside. These wear plates, whenever they become worn down 1/2" below their initial surface level, at any point of their surface, must be replaced with new. The metal of which these wear plates is to be composed shall be what is known as hard machinery iron, and must contain not less than 1 % of combined carbon. The faces of the polygon must be smooth and give uniform

[1] JUDSON, "Roads and Pavements," p. 87.

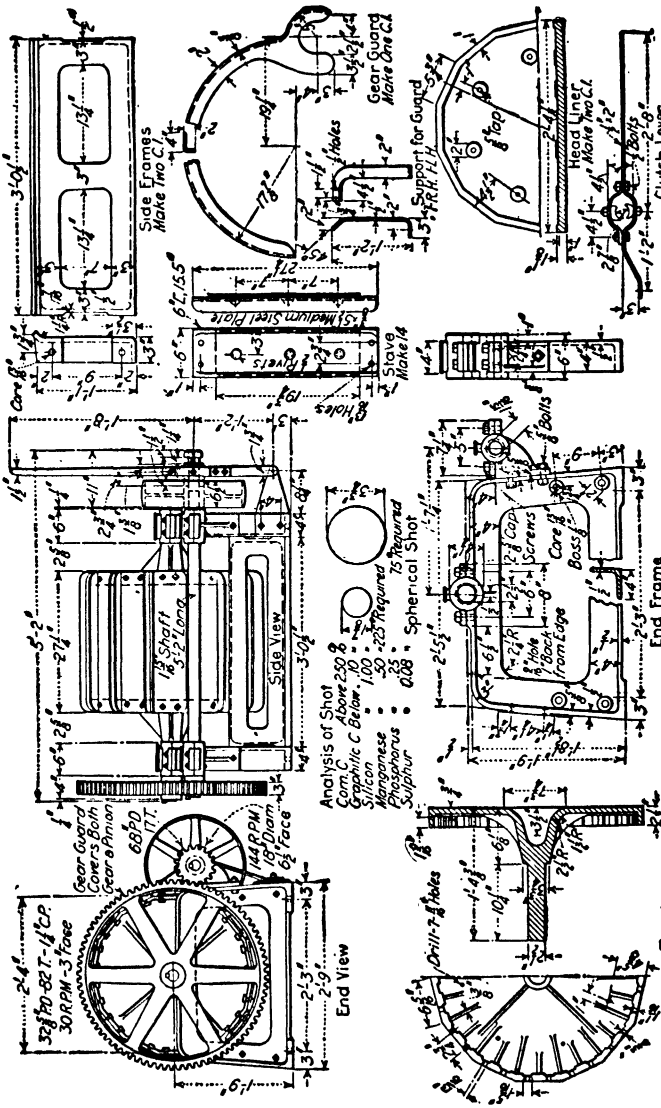


FIG. 257.—Details of brick rattler.

bearing for the staves. To secure the desired uniform bearing the faces of the head may be ground or machined.

"**The Staves.**—The staves shall be made of 6″ medium steel structural channels 27¼″ long and weighing 15.5 lb. per lineal foot.

"The channels shall be drilled with holes 13/16″ in diameter, two in each end, for bolts to fasten same to head, the center line of the holes being 1″ from either end and 1⅝″ either way from the longitudinal center line.

"The space between the staves will be determined by the accuracy of the heads, but must not exceed 5/16″. The interior or flat side of each channel must be protected by a lining or wear plate ⅜″ thick by 5½″ wide by 19¾″ long. The wear plate shall consist of medium steel plate, and shall be riveted to the channel by three ½″ rivets, one of which shall be on the center line both ways and the other two on the longitudinal center line and spaced 7″ from the center each way. The rivet holes shall be countersunk on the face of the wear plate and the rivets shall be driven hot and chipped off flush with the surface of the wear plate. These wear plates shall be inspected from time to time, and if found loose shall be at once riveted, but no wear plate shall be replaced by a new one except as the whole set is changed. No set of wear plates shall be used for more than 150 tests under any circumstances. The record must show the date each set of wear plates goes into service and the number of tests made upon each set.

"The staves when bolted to the heads shall form a barrel 20″ long inside measurement, between wear plates. The wear plates of the staves must be so placed as to drop between the wear plates of the heads. These staves shall be bolted tightly to the heads by four ¾″ bolts, and each bolt shall be provided with lock nuts and shall be inspected at not less frequent intervals than every fifth test and all nuts kept tight. A record shall be made after each inspection, showing in what conditions the bolts were found.

"**The Frame and Driving Mechanism.**—The barrel should be mounted on a cast-iron frame of sufficient strength and rigidity to support same without undue vibration. It should rest on a rigid foundation and be fastened to same by bolts at not less than four points.

"It should be driven by gearing whose ratio of driver to driven should not be less than 1:4. The counter shaft upon which the driving pinion is mounted should not be less than 1 15/16″ in diameter, with bearings not less than 18″ in diameter and 6½″ in face. A belt of 6″ double-strength leather, properly adjusted, so as to avoid unnecessary slipping, should be used.

"(As a part of this publication will be found a complete working drawing of a machine which will meet the above specifications and requirements.)

"**The Abrasive Charge.**—*a.* The abrasive charge shall consist of two sizes of cast-iron spheres. The larger size shall be 3¾″ in diameter when new, approximately 7.5 lb. (3.40 kilos) each. Ten shall be used.

"These shall be weighed separately after each ten tests, and if the weight of any large shot falls to 7 lb. (3.175 kilos) it shall be discarded and a new one substituted; provided, however, that all of the large shot shall not be discarded and substituted by a new one at any single time, and that, so far as possible, the large shots shall compose a graduated series in various stages of wear.

"The smaller-size spheres shall be, when new, 1.875″ in diameter and shall weigh not to exceed 0.95 lb. (0.430 kilo) each. Of these spheres so many shall be used as will bring the collective weight of the large and small spheres most nearly to 300 lb., provided no small sphere shall be retained in use after it has been worn down so that it will pass a circular hole 1¾″ in diameter, drilled in a cast-iron plate ¼″ in thickness, or weigh less than 0.75 lb. (or 0.34 kilos). Further, the small spheres shall be tested by passing them over such an iron plate drilled with such holes, or shall be weighed after every ten tests, and any which pass through or fall below specified weight shall be replaced by new spheres, and provided, further, that all of the small spheres shall not be rejected and replaced by new ones at any one time, and that so far as possible the small spheres shall compose a graduated series in various stages of wear. At any time that any sphere is found to be broken or defective it shall at once be replaced.

"*b.* The iron composing these spheres shall have a chemical composition within the following limits:

Combined carbon, not less than 2.50%.
Graphitic carbon, not more than 0.10%.
Silicon, not more than 1%.
Manganese, not more than 0.50%.
Phosphorus, not more than 0.25%.
Sulphur, not more than 0.08%.

"For each new batch of spheres used the chemical analysis must be furnished by the maker, or be obtained by the user, before introduction into the charge, and unless the analysis meets the above specifications the batch of spheres shall be rejected.

"The Brick Charge.—The number of brick per charge shall be ten for all bricks of the so-called "block size" whose dimensions fall between from 8 to 9″ in length, 3¾″ in breadth, and 3¾ and 4¼″ in thickness. No block should be selected for test that would be rejected by any other requirements for the specifications.

" The brick shall be clean and dried for at least 3 hrs. in a temperature of 100°F before testing,

"Speed and Duration of Revolution.—The rattler shall be rotated at a uniform rate of not less than 29½ nor more than 30½ r.p.m., and 1800 revolutions shall constitute the standard test.

"A counting machine shall be attached to the rattler for counting the revolutions. A margin of not to exceed ten revolutions will be allowed for stopping. One start and stop per test is regular and acceptable.

"The Results.—The loss shall be calculated in percentage of the original weight of the dried brick composing the charge. In weighing the rattled brick, any piece weighing less than 1 lb. shall be rejected.

"Records.—*a*. The operator shall keep an official book, in which the alternate pages are perforated for removal. The record shall be kept in duplicate, by use of a carbon paper between the first and second sheets, and when all entries are made and calculations are completed the original record shall be removed and the carbon duplicate preserved in the book. All calculations must be made in the space left for that purpose in the record blank, and the actual figures must appear. The record must bear its serial number and be filled out completely for each test, and all data as to dates of inspection and weighing of shot and replacement of worn-out parts must be carefully entered, so that the records remaining in the book are continuous. In event of further copies of a record being needed, they may be furnished on separate sheets, but in no case shall the original carbon copy be removed from the record book.

REPORT OF STANDARD RATTLER TEST OF PAVING BRICKS

Identification Data Serial (—)

Name of firm furnishing sample . . . .
Name of firm manufacturing sample. . . . .
Street or job which sample represents. . . . .
Brands or marks on the brick . . . .
Quantity furnished. . . . . Drying treatment. . . . .
Date received. . . . . Date tested . . . .
Length. . . . . Breadth. . . . . Thickness. . . . .

Standardization Data

Number of charges treated since last inspection. . . . .
Weight of charge (after standardization). . . . .
Condition of locknuts on staves. . . . .
Condition of scales. . . . .
10 Large spheres. . . . .
Small spheres. . . . .
Total. . . . .
Number of charges tested since stave linings were renewed
Repairs (note any repairs affecting the condition of the barrel)

Running Data

Time readings. . . . . . Revolution. . . . . Running notes. . . . .
Counter. . . . . Stops, etc. . . . .
Readings. . . . .
Hours. . . . . . Minutes. . . . . . Seconds. . . .
Beginning of test. . . . .
Final reading. . . . .

Weights and Calculations

Initial weight of 10 bricks.............. Percentage loss...........
Final weight of same................... (NOTE.—The calculation
Loss of weight........................ must appear.)
Number of broken bricks and remarks on same
I certify that the foregoing test was made under the specifications of .... and is a true record.
Date............ Signature of (tester)....... ....
Location of laboratory.................

"Any brick which loses 24% or more in the rattler, or increases more than 3½% in weight or less than one-half of 1% in the absorption test will be rejected."

6. BITUMINOUS BINDERS

The subject of bitumens is an intricate one and the reader is referred to the works of Clifford Richardson, Prevost Hubbard, and others for detailed information, as a book of this character can give only an outline.

There are a number of dust preventives and road binders on the market which depend for their effectiveness on a bituminous-binding base. The term "bitumen" is applied to a great many substances. Hubbard arbitrarily defines bitumens as "consisting of a mixture of native or pyrogenetic hydrocarbons and their derivatives, which may be gaseous, liquid, a viscous liquid, or solid, but if solid, melting more or less readily upon the application of heat and soluble in chloroform, carbon bisulphide, and similar solvents."[1]

The bitumens may be classed as native and artificial. The native bituminous materials, that are used in road work, are the asphaltic and semiasphaltic oils (dust layers), Malthas (the binding base of rock asphalts), Trinidad, Bermudez California, and Cuba asphalts, gilsonite, and grahamite (which, however, are too brittle in their natural state and require fluxing with a suitable residual oil before they can be used as binders). The natural asphalts are refined to remove water and any objectionable amount of impurities by heating until the gases are driven off, skimming the vegetable matter which rises to the surface, and removing the mineral constituents which fall to the bottom.

The artificial bituminous materials are derived by the destructive distillation of coal, or by fractional distillation of crude coal tars, or the native petroleum oils. They comprise the crude coal and water-gas tars, the refined tars, the residual oils and semisolid binders derived from the petroleum oils. They vary greatly in consistency and binding power.

The tests and detailed requirements for light, medium, and heavy bitumens are given on pages 727 and 1388.

The following material is briefed from *Bulletin* 34, U. S. Office of Public Roads: The light oils and tars have a relative small percentage of bituminous base and are effective only so long as it retains its binding power; the more permanent binders contain a larger percentage of bitumen; these are the heavy oils and semisolids.

[1] "Dust Preventives and Road Binders," John Wiley & Sons, Inc.

ARTIFICIAL BITUMENS

Crude Tars.—Coke ovens and gas plants produce most of the coal tars in use. These tars contain various complex combinations of carbon, hydrogen, and oxygen and small amounts of nitrogen and sulphur. They vary in composition according to the material from which they are made and the temperature at which they are distilled. The percentage of free carbon ranges from 5 to 35%, and the bitumen from 60 to 95%, depending on the temperature of manufacture. Tars produced at high temperatures contain free carbon in excess which weakens their binding power; they also contain a large amount of anthracine and naphthalene, two useless materials from the standpoint of road work. Tars produced at low temperatures are to be preferred. Coke tar is low-temperature tar; gas tar is high-temperature tar.

TABLE 122.—SPECIFIC GRAVITY AND COMPOSITION OF TAR PRODUCTS. (*From Bulletin* 34, *U. S. Office of Public Roads*)

Kind of tar	Specific gravity	Ammoniacal water, %	Total light oils to170°C., %	Total dead oils 170–270°C., %	Residue (by difference), %
Water-gas tar..........	1.041	2.4	21.6[a]	52.0[b]	24.0[c]
Crude coal tar..	1.210	2.0	7.2[d]	26.0[e]	54.8[f]
Refined coal tar........	1.177	0.0	12.8[b]	47.6[g]	39.6[f]

[a] Distillate, mostly liquid.
[b] Distillate, all liquid.
[c] Pitch, very brittle.
[d] Distillate, mostly solid.
[e] Distillate, one-half solid.
[f] Pitch, hard and brittle.
[g] Distillate, one-third solid.

Refined Tars.—Much of the road tar is refined tar—that is, it has been subjected to fractional distillation to remove the valuable volatile compounds. The residuum from this process is a thick, viscous material known as coal-tar pitch, and if the crude tar from which it is obtained was produced at a low temperature it is nearly pure bitumen; the dead oils obtained from the distillation are of little value and are often run back into the pitch, which makes it liquid when cold. The accompanying table gives the approximate composition of water-gas tar, crude coal tar, and refined tar.

If the tar is used as a temporary dust layer only, it should be low-temperature, dehydrated tar, liquid when cold. If used as a more permanent binder and applied hot, it should have a larger percentage of pitch, should contain no water, and be free from an excessive amount of free carbon. If used as a mastic in bituminous macadam, it should contain a high percentage of pitch and be free from the defects mentioned.

Natural Bitumens and Artificial Oils and Semi-solids.—Mineral oils can be classed as paraffin petroleums, mixed paraffin and asphaltic petroleums, and asphaltic petroleums. The relative value of oils as a source of supply for road materials depends on their percentage of asphaltic residue. The eastern oils found in New York, Pennsylvania, West Virginia, etc., are paraffin petroleums; the western oils vary from light to heavy asphaltic petroleums, and the southern oils have a mixed paraffin and asphaltic base.

The crude petroleum is refined by fractional distillation to obtain its valuable products, such as kerosene, etc. The character of the residue depends, as for the tars, on the crude material and the method of manufacture; the operation, known as "cracking," which is used to increase the yield of the inflammable oils, produces an excess of free carbon.

The paraffin petroleum residuums are soft and greasy and are not suitable for road work; they contain a large amount of the paraffin hydrocarbons and paraffin scale (crude paraffin).

The California petroleum residuums resemble asphalt, and if carefully distilled without cracking should contain little or no free carbon. They are suited to road work.

The Texas or semiasphaltic petroleums contain some paraffin hydrocarbons and about 1% of paraffin scale. Residuums from these oils, if containing a relatively small amount of paraffin, can be successfully used.

The required properties of residuum binders used on the New Jersey State roads in 1922 are given (p. 1388).

The accompanying tables give a general idea of the relative characteristics of the crude petroleums and petroleum residuums.

TABLE 123.—RESULTS OF TESTS OF CRUDE PETROLEUM. (*From Bulletin 34, U. S. Office of Public Roads*)

Kinds of oil	Specific gravity	Flash point, degrees centigrade	Volatility at 110°C. 7 hr., %	Volatility at 160°C. 7 hr., %	Volatility at 205°C. 7 hr., %	Residue, %
Pennsylvania, paraffin......	0.801	[a]	47.3	58.0	68.0	32.0[b]
Texas, semiasphaltic.....	0.904	43	20.0	27.0	49.0	51.0[c]
California, asphaltic.......	0.939	26	....	....	42.7[d]	57.3[e]

[a] Ordinary temperature.
[b] Soft.
[c] Quick flow.
[d] Volatility at 200°, 7 hr.
[e] Soft maltha; sticky.

TABLE 124.—RESULTS OF PETROLEUM RESIDUUM

Kinds of oil	Specific gravity	Flash point, degrees centigrade	Volatility at 200°C. 7 hr., %	Residue, %	Solid paraffin, %	Fixed carbon, %
Pennsylvania, paraffin......	0.920	186	14.2	85.8[a]	11.0	3.0
Texas, semiasphaltic.......	0.974	214	6.2	93.8[a]	1.7	3.5
California, asphaltic........	1.006	191	17.3	82.7[a]	0.0	6.0

[a] Soft.

Tests of Bitumens and Their Significance.—Bitumens for use as the cementing material in road construction may, according to their source and characteristics, be divided into the two general classes of asphalt and tars.

The asphalts suitable for use as the cementing agent in road construction are produced either by reducing asphaltic-base petroleum to a suitable consistency by the distillation process or by softening the so-called solid asphalts to a suitable consistency by the addition of flux produced by the partial distillation of petroleum.

The different grades, relative to consistency, of road oils are usually produced by the partial reduction of asphaltic-base petroleum.

By the destructive distillation of bituminous coals or the "cracking" of petroleum oils during the carburetting process in the manufacture of water gas, crude tars are produced. These crude tars are refined or reduced by distillation to a suitable consistency for use in road construction.

Bitumens are used in road construction for the purpose of waterproofing the surface and adding to the mechanical bond of the mineral aggregate by cementing together the finer particles of mineral matter, thus preventing their displacement under the action of traffic and retaining them in the road surface where they fill the interstices between the larger stone and bind them together.

The desirable characteristics of bituminous material for road-building purposes are: (1) adhesiveness, (2) non-susceptibility to changes in temperature, and (3) stability or "life." The chief object of bituminous material specifications is to make imperative these desirable qualities of the material.

In testing bituminous materials it should be remembered that the laboratory results obtained in the different tests are largely for comparative purposes. By this means new or but-little-used materials may be compared with materials which have proved satisfactory under service tests. Also laboratory results furnish an accurate means of specifying the exact characteristics of the material desired for any given purpose.

Adhesiveness.—The adhesiveness of the material is provided for in specifications by suitable requirements of ductility and toughness.

The ductility and toughness tests are made for the purpose of determining the adhesive and binding qualities of the material under different conditions of temperature. The ductility test is made by determining the distance a briquette of the material, having a cross-section 1 sq. cm. will draw before breaking. Since temperature affects the results, a standard temperature of 77°F. has been adopted generally for making this test. Experience teaches that the greater the distance that a briquette of the material will stretch out before breaking, the more sticky and adhesive the material. This test may be performed in a rough manner by pulling out a small roll of the material between the fingers. Material which will not pull out to a long thread before breaking is usually spoken of as "short." Such materials are not adhesive or sticky and it is extremely difficult to bind a road with them, even under the most favorable circumstances.

As stated, the ductility test is usually made at a temperature of 77°F., and thus measures the adhesiveness of the material at a rather high temperature. To obtain an indication of the character of the material at a low temperature the toughness test is made at a temperature of 32°F. This test is performed by dropping a weight of 2 kg. on a cylinder of the material 1¾" in diameter by 1¾" in height. The first height of the drop is usually from a distance of 5 cm. and is gradually increased until rupture of the cylinder occurs. A rough field test for toughness may be performed by noting whether a piece of the material will fracture under a sharp blow. If the temperature of the material is about 32°F., the results will be more indicative of the character of the material.

Bitumens which are brittle or which give a low toughness result, lose their binding value in cold weather and roads constructed by their use are apt to ravel and break up under traffic.

Bitumens which give good ductility and toughness results under the methods outlined will give satisfactory results as the cementing medium when used in road construction, provided the other construction details have been properly followed out.

In connection with the stickiness and adhesiveness of bitumens, the fact should always be kept in mind that their purpose in road construction as cementing medium is most effective when used with a hard, clean, dry mineral aggregate. As the departure from these qualities of the mineral aggregate increases so also the difficulties are increased of getting a satisfactory road surface firmly bound together.

Susceptibility to Changes in Temperature.—The susceptibility to changes in temperature is shown by the relative hardness as indicated by the penetration tests at different temperatures, as at 32, 77, and 115°F.

The consistency of asphalts is referred to as the "penetration." The penetration test is made by measuring the distance in hundredths of a centimeter that a standard needle under a stated load, applied for a stated time, will penetrate into it vertically. These variable factors are usually as follows:

NEEDLE: R. J. ROBERTS' PARABOLA "SHARPS" No. 2

At 32°F. 200-g. weight, 1 min.
At 77°F. 100-g. weight, 5 sec.
At 115°F. 50-g. weight, 5 sec.

The material which is the most susceptible to changes in temperature will show the greatest variation in penetration under varying conditions of temperature. Roads constructed by the use of materials which are extremely susceptible to changes in temperature become soft in warm weather, mark easily, have a tendency to rut, and become wavy. In cold weather this material becomes very hard and slippery and is apt to be brittle and become chipped from the road surface.

In addition to the general qualities of bitumens which are shown by penetration tests, this test is used in specifications to define within narrow limits the consistency of the material. The consistency limits placed in specifications are governed by the climate and the type of construction to be followed, also the general size of the mineral aggregate to be used. When the penetration method of construction is followed it is necessary to use a relatively soft asphalt in order that it may be incorporated in the road surface. In the mixing types of construction a harder asphalt may be incorporated with the mineral aggregate. The use of a hard asphalt together with a graded mineral aggregate gives a dense wearing surface that does not readily become wavy under traffic.

The information obtained by the penetration test is not readily checked in the field without the aid of laboratory apparatus, but, as a general rule, bitumens which are suitable for binders are plastic when "worked" in the hands.

Stability.—When the term "stability" or "life" is used in reference to bitumens, it refers to the quality of the material by which it retains its characteristics over a long period of time. The laboratory tests which indicate this property are the evaporation test, the ratio of the penetration after evaporation to the original penetration, and the flash point.

The heating or evaporation test is made by placing 50 g. of the material in a flat-bottomed dish 2⅜₁₆" in diameter by 1⅜" in depth. This is placed in an oven maintained at specified temperature, usually 325°F., for a period of 5 hr.

This test may be considered as an accelerated test on the material. In a binder, the percentage lost by weight together with the resulting hardening as shown by the relative penetration, *i.e.*, the ratio of the original penetration to the penetration after evaporation, are indicative of the "life" of the material. The less the evaporation loss and the less the hardening as shown by the relative penetration the greater will be the "life" of the material.

In an oil used for surface application the evaporation test shows the presence and quantity of light oils. This is indicative of the time required for the oil to "set up" after application to the road surface, the evaporation from the large-surface area of the oil as applied to the road being roughly comparable with evaporation from the smallest surface area of the oil exposed at the higher temperature at which the test is made.

The open-flash test is made by heating at the rate of about 10°F. per minute a small quantity of the material, approximately 40 g., in a dish of about the same size as the dish used for the penetration tests, 2$\frac{3}{16}$" in diameter by 1⅜" in depth. A small flame from a capillary tube is passed over the surface of the oil at each increase of 5° in temperature.

A slight "puff" or explosion indicates the flash point has been reached. The presence of light oils or distillates is indicated by a low flash point. The flash point together with the evaporation results give an indication as to the methods and materials used in the manufacture of the bitumen which is being tested.

Unless "cut-back" materials are being tested, in which an exceedingly light distillate as naphtha or benzole has been used as the "cut-back" agent, considerable "smoke" will be given off from the sample before the flash point is reached. This feature should be kept in mind when material is being heated for application in the field. Material should never be heated in the field to a point when it smokes profusely, for at such a temperature the material is being "burned" or hardened to such an extent that it loses its adhesiveness and becomes brittle when cold, thus failing to become a binding or cementing agent which binds the mineral aggregate of the road together.

The same "burning" effect on the material is produced by keeping it at a temperature below the "smoking point" for a long period (several hours) as would be produced at a higher temperature for a shorter period of time. This important feature should always be kept in mind when heating material for application in the field.

Such tests as those for water, specific gravity, purity, paraffin, etc., are usually placed in specifications in addition to the tests which govern adhesiveness, non-susceptibility, and stability for the purpose of identification of materials used, methods of manufacture, degree of refinement, and care used in refining.

The presence of water in bituminous materials causes frothing when heated to a temperature of about 212°F. In addition to the difficulty experienced in heating material containing water, due to the frothing an even application or distribution to the road of such material is extremely difficult due to the presence of the froth, which is likely to be applied rather than the liquid bitumen.

Tests for specific gravity, purity, paraffin, etc., require laboratory apparatus to get results which indicate qualities of the material. The information obtained by these tests cannot be obtained by field tests.

If it is assumed that a suitable bitumen has been specified and obtained for construction work in which a bitumen is to serve as the cementing material, the results obtained, relative to the bitumen, will depend upon:

1. Not overheating (by high temperature or long time) the bitumen.
2. The use of hard, clean, dry stone.
3. Grading of the mineral aggregate to reduce voids and obtain greater density.
4. Thorough and uniform incorporation of the bitumen with mineral aggregate.
5. Maximum consolidation, by rolling when laid.

When bituminous materials which may be applied cold are to be applied to a road surface, that surface should first be put in good condition. Surface application treatment is for the purpose of preserving a road which is in good condition and not repairing an uneven road. A house is not repaired by painting it; rather the house is repaired and then painted, in order that it may remain in good condition. An attempt to build up a road-wearing surface by the use of bitumens which may be applied cold usually results in a surface which is easily marked, rutted, and pushed into waves.

The following detailed description of "Recent Test Methods" for asphalts is quoted from Brochure 8, Asphalt Association:

Methods of Testing.—The following methods of testing are those to which reference is made in the specifications. Some of the descriptions have been condensed, but not changed in any essential particular.

SPECIFIC GRAVITY[1]

"**Pycnometer Method.**—The pycnometer (shown in Fig. 257*A*) consists of a fairly heavy, straight-walled glass tube, 70 mm. long and 22 mm. in diameter, carefully ground to receive an accurately fitting solid glass stopper with a hole of 1.6 mm. bore in place of the usual capillary opening. The lower part of this stopper is made concave in order to allow all air bubbles to escape through the bore. The depth of the cup-shaped depression is 4.8 mm. at the center. The stoppered tube has a capacity of about 24 cc. and when empty weighs about 28 g.

"The clean, dry pycnometer is first weighed empty and this weight is called *a*. It is then filled in the usual manner with freshly distilled water at 25°C., and the weight is again taken and called *b*. A small amount of the bitumen should be placed on a spoon and brought to a fluid condition by the gentle application of heat, with care that no loss by evaporation occurs. When sufficiently fluid, enough is poured into the dry pycnometer, which may also be warmed, to fill it about half full, without allowing the material to touch the sides of the tube above the desired level. The tube and contents are then allowed to cool to room temperature, after which the tube is carefully weighed with the stopper. This weight is called *c*. Distilled water at 25°C. is then poured in until the pycnometer is full. After this the stopper is inserted, and the whole cooled to 25°C. by a 30-min. immersion in a beaker of distilled water maintained at this temperature. All surplus moisture is then removed with a soft cloth, and the pycnometer and contents are weighed. This weight is called *d*. From the weights obtained the specific gravity of the bitumen may be readily calculated by the following formula:

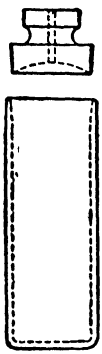


FIG. 257*A*.—Pycnometer (Hubbard type).

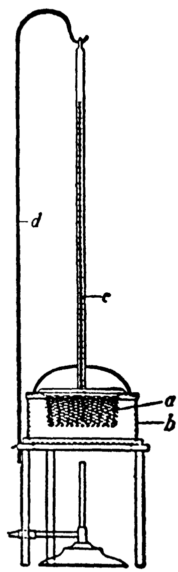


FIG. 257*B*.—Open-cup oil tester.

$$\text{Specific gravity } 25°\text{C.}/25°\text{C.} = \frac{c - a}{(b - a) - (d - c)}.$$

"The specific gravity of fluid bitumen may be determined in the ordinary manner with this pycnometer by completely filling it with the material and dividing the weight of the bitumen thus obtained by that of the same volume of water."

FLASH POINT[2]

"**Open-cup Method.**—The open-cup oil tester (shown in Fig. 257*B*), consists of a brass oil cup *a* of about 100-cc. capacity, and an outer vessel *b*

[1] *U. S. Dept. Agr., Bull.* 314, p. 5.
[2] *U. S. Dept. Agr., Bull.* 314, p. 17.

which serves as an air jacket; a suitable thermometer *c* is suspended from the wire support *d* directly over the center of the cup so that its bulb is entirely covered with asphalt but does not touch the bottom of the cup. The testing flame is obtained from a jet of gas passed through a piece of glass tubing, and should be about 5 mm. in length.

"The test is made by first filling the oil cup with the material under examination to within about 5 mm. of the top. The Bunsen flame is then applied in such a manner that the temperature of the material in the cup is raised at the rate of 5°C. per minute. From time to time the testing flame is brought almost in contact with the surface of the asphalt. A distinct flicker or flash over the entire surface of the asphalt shows that the flash point is reached and the temperature at this point is taken."

PENETRATION[1]

"**I. Definition.** 1. *Penetration.*—Penetration is defined as the consistency of a bituminous material expressed as the distance that a standard needle vertically penetrates a sample of the material under known conditions of loading, time, and temperature. When the conditions of test are not specifically mentioned, the load, time, and temperature are understood to be 100 g., 5 sec., 25°C. (77°F.), respectively, and the units of penetration to indicate hundredths of a centimeter.

"**II. Apparatus.** 2. *Container.*—The container for holding the material to be tested shall be a flat-bottom, cylindrical dish, 55 mm. (2³⁄₁₆″) in diameter and 35 mm. (1⅜″) deep.[2]

"3. *Needle.*—The needle[3] for this test shall be of cylindrical steel rod 50.8 mm. (2″) long and having a diameter of 1.016 mm. (0.04″) and turned on one end to a sharp point having a taper of 6.35 mm. (¼″).[4]

"4. *Water Bath.*—The water bath shall be maintained at a temperature not varying more than 0.1°C. from 25°C. (77°F.). The volume of water shall not be less than 10l and the sample shall be immersed to a depth of not less than 10 cm. (4″) and shall be supported on a perforated shelf not less than 5 cm. (2″) from the bottom of the bath.

"5. *Apparatus for Penetration.*—Any apparatus which will allow the needle to penetrate without appreciable friction, and which is accurately calibrated to yield results in accordance with the definition of penetration will be acceptable.

"6. *Transfer Dish for Container.*—The transfer dish for container shall be a small dish or tray of such capacity as will insure complete immersion of the container during the test. It shall be provided with some means which will insure a firm bearing and prevent rocking the container.

"**III. Preparation of Sample.** 7. *Preparation of Sample.*—The sample shall be completely melted at the lowest possible temperature and stirred thoroughly until it is homogeneous and free from air bubbles. It shall then be poured into the sample container to a depth of not less than 15 mm. (⅝″). The sample shall be protected from dust and allowed to cool in an atmosphere not lower than 18°C. (65°F.) for 1 hr. It shall then be placed in the water bath along with the transfer dish and allowed to remain 1 hr.

"**IV. Testing.** 8. *Testing.*—*a.* In making the test the sample shall be placed in the transfer dish filled with water from the water bath of sufficient depth to cover the container completely. The transfer dish containing the sample shall then be placed upon the stand of the penetration machine. The needle, loaded with specified weight, shall be adjusted to make contact with the surface of the sample. This may be accomplished by making contact of the actual needle point with its image reflected by the surface of the sample from a properly placed source of light. Either the reading of the dial shall then be noted or the needle brought to zero. The needle is then released for the specified period of time, after which the penetration machine is adjusted to measure the distance penetrated.

"At least three tests shall be made at points on the surface of the sample not less than 1 cm. (⅜″) from the side of the container and not less than 1 cm. (⅜″) apart. After each test the sample and transfer dish shall be returned to the water bath and the needle shall be carefully wiped toward

[1] Amer. Soc. Testing Materials, Standard Test D5-16.

[2] This requirement is fulfilled by the American Can Co.'s gill-style ointment box, deep pattern, 3-oz. capacity.

[3] *J. Agr. Research*, vol. 5, 24, pp. 1125–1126.

[4] No. 2 Roberts sewing needle which has been carefully checked with a standard needle may be used.

its point with a clean, dry cloth to remove all adhering bitumen. The reported penetration shall be the average of at least three tests whose values shall not differ more than four points between maximum and minimum.

"*b.* When desirable to vary the temperature, time, and weight and in order to provide for a uniform method of reporting results when variations are made, the samples shall be melted and cooled in air as above directed. They shall then be immersed in water or brine, as the case may require, for 1 hr. at the temperature desired. The following combinations are suggested:

"At 0°C. (32°F.), 200-g. weight, 60 sec.

"At 46.1°C. (115°F.), 50-g. weight, 5 sec."

DUCTILITY[1]

"A briquette of the material to be tested shall be formed by pouring the molten material into a briquette mold. The dimensions of the briquette shall be: 1 cm. (0.394") in thickness throughout its entire length; distance between the clips or end pieces, 3 cm. (1.181"); width of asphalt-cement section at mouth of clips, 1 cm. (0.394"). The center pieces are removable, the briquette mold being held together during molding with a clamp or wire.

"The molding of the briquette shall be done as follows: The two center sections shall be well amalgamated to prevent the asphalt cement from adhering to them, and the briquette mold shall then be placed on a freshly amalgamated brass plate. The asphalt cement to be tested, while in a molten state, shall be poured into the mold, a slight excess being added to allow for shrinkage on cooling. When the asphalt cement in the mold is nearly cool, the briquette shall be cut off level, with a warm knife or spatula. When it is thoroughly cooled to the temperature at which it is desired to make the test, the clamp and the two side pieces are removed, leaving the briquette of asphalt cement held at each end by the ends of the mold, which now play the part of clips. The briquette shall be kept in water for 30 min. at 4°C. (39°F.) or 25°C. (77°F.) before testing, dependent on the temperature at which the ductility is desired. The briquette and the clips attached shall then be placed in a ductility-test machine filled with water at one of the above temperatures to a sufficient height to cover the briquette not less than 50 mm. (1.969"). This machine consists of a rectangular water-tight box, having a movable block working on a worm-gear from left to right. The left clip is held rigid by placing its ring on a short metal peg provided for this purpose; the right clip is placed over a similar rigid peg on the movable block. The movable block is provided with a pointer which moves along a centimeter scale. Before starting the test, the centimeter scale is adjusted to the pointer at zero. Power is then applied by the worm gear pulling from left to right at the uniform rate of 5 cm. (1.969") per minute. The distance, in centimeters, registered by the pointer on the scale at the time of rupture of the thread of asphalt cement shall be taken as the ductility of the asphalt cement."

VOLATILIZATION TEST[2]

"The oven shown in Fig. 257*C*, known as the New York testing laboratory oven, is used by the Bureau of Public Roads, although any other form may be used that will give a uniform temperature throughout all parts where samples are placed. The bulb of one of the thermometers is immersed in a sample of some fluid, non-volatile bitumen, while the other is kept in air at the same level. The first thermometer serves to show the temperature of the samples during the test, while the latter gives prompt warning of any sudden changes in temperature due to irregularities in the gas pressure, etc.

"Before making the test the interior of the oven should show a temperature of 163°C. as registered by the thermometer in air. A round tin box 5½ cm. in diameter and 3½ cm. deep is accurately weighed after carefully wiping with a towel to remove any grease or dirt. About 50 g. of the material to be tested is then placed in the box. The material may then be weighed on a rough balance, if one is at hand, after which the accurate weight, which should not vary more than 0.2 g. from the specified amount, is obtained. It may be necessary to warm some of the material in order to handle it con-

[1] *Trans. Am. Soc. Civil Eng.*, vol. 82, p. 1460, 1918.

[2] *U. S. Dept. Agr., Bull.* 314, p. 19.

veniently, after which it must be allowed to cool before determining the accurate weight.

"The sample should now be placed in the oven, where it is allowed to remain for a period of 5 hr., during which time the temperature as shown by the thermometer in bitumen should not vary at any time more than 2°C. from 163°C. The sample is then removed from the oven, allowed to cool, and reweighed. From the difference between this weight and the total weight before heating the percentage of loss on the amount of material taken is calculated.

"Highly volatile and non-volatile materials should not be subjected to this test at the same time in the same oven, owing to a tendency on the part of the latter to absorb some of the volatile products of the former.

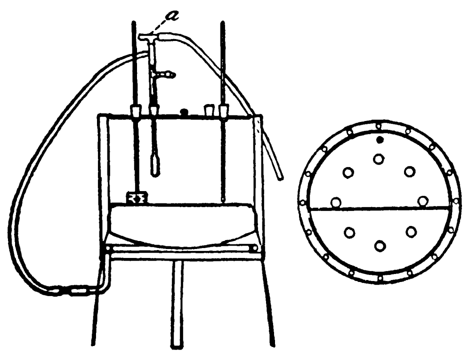


FIG. 257*C*.—New York testing laboratory oven.

"Some relative idea of the amount of hardening which has taken place may be obtained from the results of a float or penetration test made on the residue, as compared with the results of the same tests on the original sample. Before any tests are made on the residue, it should be melted and thoroughly stirred while cooling."

TOTAL BITUMEN[1]

"**Gooch Crucible Method.**—This test consists in dissolving the bitumen in carbon disulphide and recovering any insoluble matter by filtering the solution through an asbestos felt. The form of Gooch crucible best adapted for the determination is 4.4 cm. wide at the top, tapering to 3.6 cm. at the bottom, and is 2.5 cm. deep.

"For preparing the felt, the necessary apparatus is arranged as shown in Fig. 4, in which *a* is the filtering flask, *b* a rubber stopper, *c* the filter tube, and *d* a section of rubber tubing which tightly clasps the Gooch crucible *c*. The asbestos is cut with scissors into pieces not exceeding 1 cm. in length, after which it is shaken up with just sufficient water to pour easily. The crucible is filled with the suspended asbestos, which is allowed to settle for a few moments. A light suction is then applied to draw off all the water and leave a firm mat of asbestos in the crucible. More of the suspended material is added, and the operation is repeated until the felt is so dense that it scarcely transmits light when held so that the bottom of the crucible is between the eye and the source of light. The felt should then be washed several times with water and drawn firmly against the bottom of the crucible by an increased suction. The crucible is removed to a drying oven for a few minutes, after which it is ignited at red heat over a Bunsen burner, cooled in a desiccator, and weighed.

"From 1 to 2 g. of the bituminous material is now placed in the Erlenmeyer flask, which has been previously weighed, and the accurate weight of the sample is obtained. One hundred cubic centimeters of chemically

[1] *U. S. Dept. Agr., Bull.* 314, p. 25.

pure carbon disulphide is poured into the flask in small portions, with continual agitation, until all lumps disappear and nothing adheres to the bottom. The flask is then corked and set aside for 15 min.

"After being weighed, the Gooch crucible containing the felt is set up over the dry pressure flask, as shown in the figure, and the solution of bitumen in carbon disulphide is decanted through the felt without suction by gradually tilting the flask, with care not to stir up any precipitate that may have settled out. At the first sign of any sediment coming out, the decantation is stopped and the filter allowed to drain. A small amount of carbon disulphide is then washed down the sides of the flask, after which the precipitate is brought upon the felt and the flask scrubbed, if necessary, with a feather or 'policeman,' to remove all adhering material. The contents of the crucible are washed with carbon disulphide, until the washings run colorless. Suction is then applied until there is practically no odor of carbon disulphide in the crucible, after which the outside of the crucible is cleaned with a cloth moistened with a small amount of the solvent. The crucible and contents are dried in a hot-air oven at 100°C. for about 20 min., cooled in a desiccator, and weighed. If any appreciable amount of insoluble matter adheres to the flask, it should also be dried and weighed, and any increase over the original weight of the flask should be added to the weight of insoluble matter in

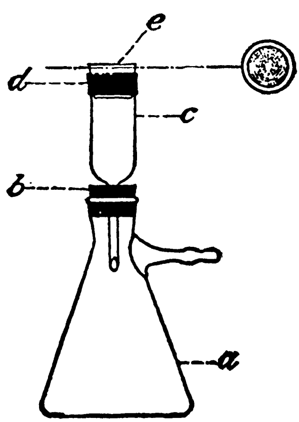


FIG. 257*D*.—Apparatus for determining soluble bitumen.

the crucible. The total weight of insoluble material may include both organic and mineral matter. The former, if present, is burned off by ignition at a red heat until no incandescent particles remain, thus leaving the mineral matter or ash, which can be weighed on cooling. The difference between the total weight of material insoluble in carbon disulphide and the weight of substance taken equals the total bitumen, and the percentage weights are calculated and reported as total bitumen, and organic and inorganic matter insoluble, on the basis of the weight of material taken for analysis.

"This method is quite satisfactory for most asphalts, but where certain lake asphalts are present it will be found practically impossible to retain all of the finely divided mineral matter on an asbestos felt. It is, therefore, generally more accurate to obtain the result for total mineral matter by direct ignition of a 1-g. sample in a platinum crucible. The total bitumen is then determined by deducting from 100% the sum of the percentages of total mineral matter and organic matter insoluble. If the presence of a carbonate mineral is suspected, the percentage of mineral matter may be most accurately obtained by treating the ash from a fixed-carbon determination with a few drops of ammonium carbonate solution, drying at 100°C., then heating for a few minutes at a dull-red heat, cooling, and weighing again.

When difficulty in filtering is experienced—for instance, when Trinidad asphalt is present in any amount—a period of longer subsidence than 15 min. is necessary, and the following method proposed by the Committee on Standard Tests for Road Materials of the American Society for Testing Materials is recommended.[1]

"From 2 to 15 g. (depending on the richness in bitumen of the substance) is weighed into a 150-cc. Erlenmeyer flask, the tare of which has been previously ascertained, and treated with 100 cc. of carbon disulphide. The flask is then loosely corked and shaken from time to time until practically all large particles of the material have been broken up, when it is set aside and not disturbed for 48 hr. The solution is then decanted off in a similar flask that has been previously weighed, as much of the solvent being poured off as possible without disturbing the residue. The first flask is again treated with fresh carbon disulphide and shaken as before, when it is put away with the second flask and not disturbed for 48 hr.

"At the end of this time the contents of the two flasks are carefully decanted off upon a weighed Gooch crucible fitted with an asbestos filter, the contents of the second flask being passed through the filter first. The asbestos filter shall be made of ignited long-fiber amphibole, packed in the bottom of a Gooch crucible to the depth of not over ⅛". After passing the contents of both flasks through the filter, the two residues are shaken with more fresh carbon disulphide and set aside for 24 hr. without disturbing, or until it is seen that a good subsidation has taken place, when the solvent is again decanted off upon the filter. This washing is continued until the filtrate or washings are practically colorless.

"The crucible and both flasks are then dried at 125°C. and weighed. The filtrate containing the bitumen is evaporated, the bituminous residue burned, and the weight of the ash thus obtained added to that of the residue in the two flasks and the crucible. The sum of these weights deducted from the weight of substance taken gives the weight of bitumen extracted."

BITUMEN SOLUBLE IN CARBON TETRACHLORIDE[2]

"**Gooch Crucible Method.**—This determination is made in exactly the same manner as described under Total Bitumen, using carbon tetrachloride as a solvent instead of carbon disulphide.

"The percentage of bitumen soluble is reported upon the basis of total bitumen taken as 100. Thus if the percentage soluble in carbon tetrachloride should be found to be 98.5% and the percentage soluble in carbon disulphide 99.0% the percentage of total bitumen soluble in carbon tetrachloride would be

$$\frac{98.5}{99.0} = 99.6\%."$$

MELTING POINT[3]

"**Ring and Ball Method.**—1. The softening of bituminous materials generally takes place at no definite moment or temperature. As the temperature rises, they gradually and imperceptibly change from a brittle or exceedingly thick and slow-flowing material to a softer and less viscous liquid. For this reason the determination of the softening point must be made by a fixed, arbitrary, and closely defined method if the results obtained are to be comparable.

"**I. Apparatus.**—2. The apparatus shall consist of the following:

"*Ring.*—*a.* A brass ring 15.875 mm. (⅝") inside diameter and 6.35 mm. (¼") deep; thickness of wall, 2.38 mm. (3/32"); permissible variation on inside diameter and thickness of ring, 0.25 mm. (0.01"). This ring shall be attached in a convenient manner to a No. 15 B. & S. gage brass wire (diameter 1.79 mm. = 0.0703") (see Fig. 257*E*).

"*b.* A steel ball 9.53 mm. (⅜") in diameter weighing between 3.45 and 3.55 g.

[1] *Proc. Am. Soc. Testing Materials*, vol. 9, p. 221, 1909.
[2] *U. S. Dept. Agr., Bull.* 314, p. 30.
[3] Am. Soc. Testing Materials, Standard Method D36-10

"*Container.*—*c.* A glass vessel, capable of being heated, not less than 9 cm. (3.54") in diameter by 13 cm. (5.12") deep. (A 600-cc. beaker. Griffin low form meets this requirement.)

"*Thermometer.*—*d.* A thermometer which shall conform to the following specifications:

Total length, 370–400 mm. (14.57–15.75").
Diameter, 6.5–7.5 mm. (0.256–0.295").
Bulb length, not over 14 mm. (not over 0.55").
Bulb diameter, 4.5–5.5 mm. (0.177–0.217").

"The scale shall be engraved upon the stem of the thermometer, shall be clear cut and distinct, and shall run from 0 to 80°C. (32 to 176°F.) in ⅕°C

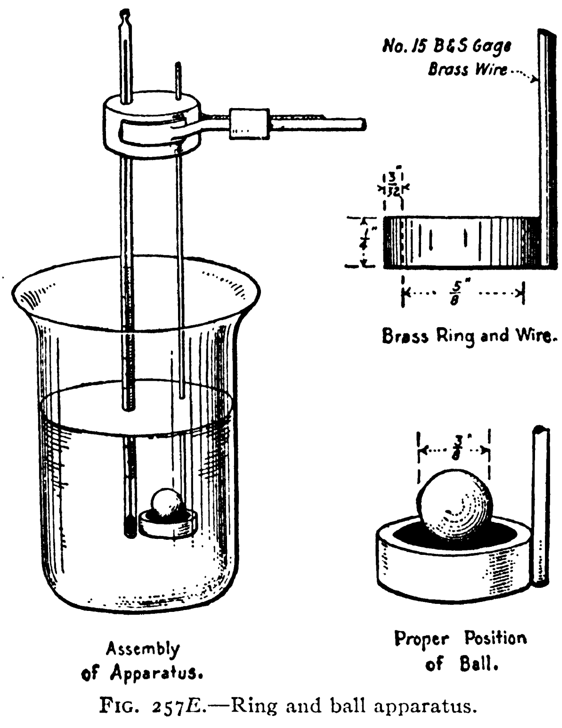


FIG. 257*E*.—Ring and ball apparatus.

divisions. It shall commence not less than 7.5 cm. (2.95 in.) above the bottom of the bulb. The thermometer shall be furnished with an expansion chamber at the top and have a ring for attaching tags. It shall be made of a suitable quality of glass and be so annealed as not to change its readings under conditions of use. It shall be correct to 0.25°C. (0.45°F.) as determined by comparison at full immersion with a similar thermometer calibrated at full immersion by the U. S. Bureau of Standards.

Preparation of Sample

"**II. Preparation of Sample.**—3. The sample shall be melted and stirred thoroughly, avoiding incorporating air bubbles in the mass, and then poured into the ring so as to leave an excess on cooling. The ring, while being filled, should rest on a brass plate which has been amalgamated to prevent the bituminous material from adhering to it. After cooling, the excess material should be cut off cleanly with a slightly heated knife.

"**III. Testing.**—*a.* Bituminous materials having softening points 90°C. (194°F.) or below.

"*Assembling.*—4. Assemble the apparatus as shown in Fig. 5. Fill the glass vessel to a depth of substantially 8.25 cm. (3.25″) with freshly boiled, distilled water at 5°C. (41°F.). Place the ball in the center of the upper surface of the bitumen in the ring and suspend it in the water so that the lower surface of the filled ring is exactly 2.54 cm. (1″) above the bottom of the glass vessel and its upper surface is 5.08 cm. (2″) below the surface of the water. Allow it to remain in the water for 15 min. before applying heat. Suspend the thermometer so that the bottom of the bulb is level with the bottom of the ring and within 0.635 cm. (¼″), but not touching, the ring.

"*Heating.*—5. Apply the heat in such a manner that the temperature of the water is raised 5°C. (9°F.) each minute.

"*Softening Point.*—6. The temperature recorded by the thermometer at the instant the bituminous material touches the bottom of the glass vessel shall be reported as the softening point.

"*Permissible Variation in Rise of Temperature.*—7. The rate of rise of temperature shall be uniform and shall not be averaged over the period of the test. The maximum permissible variation for any minute period after the first three shall be ±0.5°C. (0.9°F.). All tests in which the rate of rise in temperature exceeds these limits shall be rejected.

"*b.* Bituminous materials having softening points above 90°C. (194°F.).

"*Modification for Hard Materials.*—8. Use the same method as given under (*a*) except that glycerin shall be used instead of water.

"**IV. Accuracy.**—9. The limit of accuracy of the test is ±0.5°C. (0.9°F.)

"**V. Precautions.**—10. The use of freshly boiled, distilled water is essential, as otherwise air bubbles may form on the specimen and affect the accuracy of the results. Rigid adherence to the prescribed rate of heating is absolutely essential to secure accuracy of results.

"A sheet of paper placed on the bottom of the glass vessel and conveniently weighted will prevent the bituminous material from sticking to the glass vessel, thereby saving considerable time and trouble in cleaning."

7. CEMENT

There are five different classes of cement—Portland, natural, Puzzolan, iron-ore, and magnesia cements. Of these the Portland is usually specified.

Portland cement is the term applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3% has been made subsequent to calcination.[1]

Natural cement is the term applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.[2]

Portland cements are usually heavier, stronger, slower setting, and more uniform than the natural cements and are generally used for road structures, such as culverts, retaining walls, etc. Portland cement is practically the only cement used to any extent in the United States at the present time. The few manufacturers of natural cement who were retaining a hold on the market some few years back when the production of Portland cement was expensive are finding it difficult to compete with this latter product at its present price and quality.

[1] Am. Soc. Testing Materials, p. 353, 1915.
[2] *Ibid.*, p. 352, 1915.

The following is the standard specification for Portland cement as adopted by the American Society of Civil Engineers and the American Society for Testing Materials:

"**First: Specific Gravity.**—The specific gravity of cement shall not be less than 3.10. Should the test of cement as received fall below this requirement, a second test may be made upon a sample ignited at a low red heat. The loss in weight of the ignited cement shall not exceed 4%.

"**Second: Fineness.**—It shall leave by weight a residue of not more than 8 % on the No. 100, and not more than 25% on the No. 200 sieve.

"**Third: Time of Setting.**—It shall not develop initial set in less than 30 min.; and must develop hard set in not less than 1 hr. nor more than 10 hr.

"**Fourth: Tensile Strength.**—The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

NEAT CEMENT

Age	Strength, pounds
24 hr. in moist air	175
7 days (1 day in moist air, 6 days in water)	500
28 days (1 day in moist air, 27 days in water)	600
1 PART CEMENT 3 PARTS STANDARD OTTAWA SAND	
7 days (1 day in moist air, 6 days in water)	200
28 days (1 day in moist air, 27 days in water)	275

"**Fifth: Constancy of Volume.**—Pats of neat cement about 3″ in diameter, ½″ thick at the center, and tapering to a thin edge shall be kept in moist air for a period of 24 hr.

"*a.* A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

"*b.* Another pat is kept in water maintained as near 70°F. as practicable, and observed at intervals for at least 28 days.

"*c.* A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for 5 hr.

"These pats, to pass the requirements satisfactorily, shall remain firm and hard, and show no signs of distortion, checking, cracking, or disintegrating.

"**Sixth: Chemical Composition.**—The cement shall not contain more than 1.75% of anhydrous sulphuric acid (SO_3), nor more than 4% of magnesia (MgO)."

The equipment used in testing cement is standardized in detail and can be obtained in the Year Book published by the American Society for Testing Materials or Committee Report on Uniform Tests of Cement of the American Society of Civil Engineers, 1922.

CONCRETE MATERIALS. 8. FINE AGGREGATE

Fine aggregate for use in cement or asphaltic concrete should consists of sand free from any deleterious matter. Any sand which shows a coating on the grains should not be used until satisfactorily cleansed by washing.

The following tests are made on sand to determine its suitability for use in different classes of concrete:

1. Gradation.[1]
2. Percentage of voids.
3. Percentage of loam or silt.
4. Organic impurities.
5. Resistance to abrasion.
6. Compressive or tensile strength in cement mortar.

Mechanical Analysis of Aggregate.

TABLE 125.—AMERICAN SOCIETY FOR TESTING MATERIALS STANDARD SIEVES

Mesh designation	Unit of measure	Actual mesh	Opening	Wire diameter	Permissible variations	
					Mesh	Diameter
10[a]	Centimeter	3.9	0 200	0.056	0.04	0.005
	Inches	9.9	0.079	0.022	0 1	0.002
20[a]	Centimeter	8.0	0.085	0.040	0 2	0.0015
	Inches	20.3	0.0335	0.0157	0.5	0.0006
30	Centimeter	12.0	0.050	0 033	0.4	0.0012
	Inches	30 5	0.0197	0 0130	1.0	0 0005
40[a]	Centimeter	16.0	0.036	0.026	0.6	0.0010
	Inches	40 6	0.0142	0.0102	1.5	0 0004
50	Centimeter	20.0	0.029	0.021	0.8	0 0010
	Inches	50.8	0.0114	0.0083	2.0	0.0004
80	Centimeter	31.0	0.017	0.015	1.0	0.0008
	Inches	78.7	0.0067	0.0059	3.0	0.0003
100[a]	Centimeter	39.0	0.014	0.0116	1.0	0.0008
	Inches	99.1	0 0055	0.0046	3.0	0.0003
200[a]	Centimeter	79.0	0 0074	0.0053	3 0	0.0005
	Inches	200.7	0.0029	0.0021	8.0	0.0002

[a] It is recommended that for routine tests except for fine aggregate for hot, mixed, bituminous surfaces, the ¼" screen and these sieves be used.

STANDARD METHOD FOR MAKING A MECHANICAL ANALYSIS OF SAND OR OTHER FINE HIGHWAY MATERIAL[2]

Adopted, 1919. Revised 1916, 1918.

"The method shall consist of drying at not over 110°C. (230°F.) to a constant weight of a sample weighing of from 100 to 500 g.; passing the sample through each of the mesh sieves (American Society for Testing Materials standard sieves) specified in Table 125;[3] determining the percentage by weight retained on each sieve, the sifting being continued on each sieve until less than 1% of the weight retained on each sieve shall pass through the sieve during the last minute of sifting; and recording the mechanical analysis in the following manner:

Passing 200-mesh sieve.. ...... %
Passing 100- and retained on a 200-mesh sieve................ ...... %
Passing 80- and retained on a 100-mesh sieve.................. ...... %
Passing 50- and retained on a 80-mesh sieve.................... ...... %

100.00%"

[1] For the effect of size and gradation on strength of concrete see pp. 479 to 484, Abram's "Fineness Modulus."

[2] Amer. Soc. Testing Materials, Standard Method, Serial Designation: D7-18, slightly modified.

[3] NOTE.—The order in which the sieves are to be used in the process of sifting is immaterial and shall be left optional, but in reporting results the order in which the sieves have been used shall be stated.

MAKING A MECHANICAL ANALYSIS OF MIXTURES OF SAND OR OTHER FINE MATERIAL WITH BROKEN STONE, BROKEN SLAG, OR GRAVEL PEBBLES[1]

"The method shall consist of drying at not over 110°C. (230°F.) to a constant weight a sample weighing in pounds six times the diameter in inches of the largest holes required; separating the sample by the use of a screen having circular openings 0.64 cm. (¼") in diameter; examining the portion retained on the screen in accordance with the Standard Method for Making a Mechanical Analysis of Broken Stone or Broken Slag, except for Aggregates Used in Cement Concrete (Serial Designation: D18) of the American Society for Testing Materials; examining the portion passing this screen in accordance with the Standard Method for Making a Mechanical Analysis of Sand or Other Fine Highway Material, except for Fine Aggregates of Sand or Other Fine Highway Material, except for Fine Aggregates Used in Cement Concrete (Serial Designation D7) of the American Society for Testing Materials; and recording the mechanical analysis in the following manner:

Passing 200-mesh sieve	...... %
Passing 100- and retained on a 200-mesh sieve	...... %
Passing 80- and retained on a 100-mesh sieve	...... %
Passing 20- and retained on 80-mesh	...... %
Passing 10- and retained on a 20-mesh sieve	...... %
Passing 0.64-cm. (¼") and retained on a 10-mesh sieve	...... %
Passing 1.27-cm. (½") and retained on a 0.64-cm. (¼") screen	...... %
Passing 1.90-cm. (¾") and retained on a 1.27-cm. (½") screen	...... %
	100.00%"

Tests for Loam. *Field Test.*—It is convenient to have a graduated tube but any tall bottle will do. Fill bottle or tube about one-third or half full of sand to be tested. Add water till bottle or tube is nearly full. Stopper the top, turn into a horizontal position and shake vigorously. Change quickly to vertical position and leave for 24 hr. without disturbing. The clay and loam will settle on top in more or less clearly defined degree, the amount of which can be easily measured. This method is sufficiently close for check field tests. The Standard laboratory test is described below.

"**Test for Amount of Clay and Silt in Sand or Fine Aggregate.**—The sample as received shall be moistened and thoroughly mixed, then dried to constant weight at a temperature between 100°C. (212°F.) and 110°C. (230°F.).

"Five hundred grams representative of the dried sample shall be placed in a dried and accurately weighed pan or vessel having vertical sides and provided with a pouring lip. This pan shall be substantially 22.9 cm. (9") in diameter by not less than 10.2 cm. (4") deep. Pour sufficient water in the pan to cover the same (about 225 cc.). Agitate vigorously for 15 sec. and then pour off the water into a tared evaporating dish, taking care not to pour off any sand. Repeat until the wash is clear, using a glass rod to stir the material for the last few washings.

"Thoroughly dry the pan and washed sand in an oven at between 100°C. (212°F.) and 110°C. (230°F.), weigh and determine net weight of sand.

"Compute the per cent of clay and silt as follows:

$$\frac{\text{Original weight} - \text{weight after washing}}{\text{Original weight}} \times 100 = \text{per cent of clay and silt.}$$

"For a check on the results, evaporate the wash water to dryness and weigh the residue.

$$\frac{\text{Weight of residue}}{\text{Original weight}} \times 100 = \text{per cent of clay and silt.}$$

[1] Amer. Soc. Testing Materials, Standard Method, Serial Designation: D19-16.

"**Test for Amount of Clay and Silt in Gravel.**—The sample as received shall be moistened and thoroughly mixed, then dried to constant weight at a temperature between 100°C. (212°F.) and 110°C. (230°F.).

"A representative portion of the dry material, weighing not less than fifty times the weight of the largest stone in the sample, shall be selected from the sample and placed in a dried and accurately weighed pan or vessel. The pan shall be 30.2 cm. (12″) in diameter by not less than 10.2 cm. (4″) deep, so nearly as may be obtained. Pour sufficient water in the pan to cover the gravel and agitate vigorously for 15 sec., using a trowel or stirring rod. Allow to settle for 15 sec., and then pour off the water into a tared evaporating dish, being careful not to pour off any sand. Repeat until the wash water is clear.

"Dry the washed material to constant weight in an oven at between 100°C. (212°F.) and 110°C. (230°F.), weigh and determine net weight of gravel.

"Compute the per cent of clay and silt as follows:

$$\frac{\text{Original weight} - \text{weight after washing}}{\text{Original weight}} \times 100 = \text{per cent of clay and silt."}$$

STANDARD METHOD OF TEST FOR ORGANIC IMPURITIES IN SANDS FOR CONCRETE. SERIAL DESIGNATION: C40-22

"This method is issued under the fixed designation C40; the final number indicates the year of original adoption as standard, or in the case of revision, the year of last revision. Proposed as tentative, 1921; adopted, 1922.

"**1. Scope.**—The test herein specified is an approximate test for the presence of injurious organic compounds in natural sands for cement mortar or concrete. The principal value of the test is in furnishing a warning that further tests of the sand are necessary before they be used in concrete. Sands which produce a color in the sodium hydroxide solution darker than the standard color should be subjected to strength tests in mortar or concrete before use.

"**2. Sample.**—*a.* A representative test sample of sand of about 1 lb. shall be obtained by quartering or by the use of a sampler.

Procedure.—*b.* A 12-oz. graduated glass prescription bottle shall be filled to the 4-oz. mark with the sand to be tested.

"*c.* A 3% solution of sodium hydroxide ($NaOH$) in water shall be added until the volume of sand and liquid after shaking gives a total volume of 7 liquid oz.

"*d.* The bottle shall be stoppered and shaken thoroughly and then allowed to stand for 24 hr.

"*e.* A standard color solution shall be prepared by adding 2.5 cc. of a 2% solution of tannic acid in 10% alcohol to 22.5 cc. of a 3% sodium hydroxide solution. This shall be placed in a 12-oz. prescription bottle, stoppered and allowed to stand for 24 hr., then 25 cc. of water added.

"**Color Value.**—*f.* The color of the clear liquid above the sand shall be compared with the standard color solution prepared as in Par. *e* or with a glass of color similar to the standard solution.

"3. Solutions darker in color than the standard color have a 'color value' higher than 250 parts per million in terms of tannic acid."

"**Color Values.**—While it is not practicable to give exact values for the reduction in strength corresponding to the different colors of solution, the tests made thus far show this relation to be about as follows:

Color number (actual color prints can be obtained from Cement Association)	Reduction in compressive strength of 1.3 mortar at 7 and 28 days, %
Fig. 1	None
Fig. 2	10– 20
Fig. 3	15– 30
Fig. 4	25– 50
Fig. 5	50–100

"**Washing dirty sands has the effect of greatly reducing the quantity of organic impurities. Even after washing, however, sands should be examined in order to determine whether the organic impurities have been reduced to harmless proportions.**

"The following list includes sufficient apparatus for making five field tests at a time:

"Five 12-oz. graduated prescription bottles; stock of 3% solution of sodium hydroxide (dissolve 1 oz. of sodium hydroxide in enough water to make 32 oz.).

"This test does not give satisfactory results when lignite is present in the sand. Lignite is a mineral coal of recent geological origin. It is known commonly as 'brown or sud' coal."

Water Tests.—Water must be clean and pure, free from organic impurities or oily content.

The usual tests are for:

1. Organic impurities (see p. 738).
2. Alkaline or acid reactions. This is determined by the common litmus-paper test, and if a strong acid or alkaline reaction is shown the water should be rejected.

Proposed Abrasion for fine Aggregate.—The following is suggested as a tentative method[1] for determining the resistance ot the fine aggregate to abrasion.

"The fine aggregate is washed and dried at a temperature not exceeding 110°C. All material retained on the ¼" sieve and all material passing a standard 50-mesh sieve is discarded. Five hundred grams of the portion passing a ¼" screen and retained on a 50-mesh sieve are placed in a Deval abrasion cylinder with a charge of 250 g. of 9⁄16" commercial steel bearing balls (21 balls weigh practically 250 g.). The weight of the balls is to be within 1% of the required 250 g. The charge in the Deval abrasion cylinder is rotated for 2000 revolutions at the rate of 33 r.p.m. The sample of sand is removed and sieved over a 100-mesh sieve. The sample is preferably divided in three portions for sieving, the sieving being completed over a white sheet of paper, and is continued until practically no dust passes the sieve when shaken for 1 min. The portion retained on the 100-mesh sieve is weighed. Five hundred grams, minus the weight of the samples retained on the 100-mesh sieve after abrasion, is taken as the loss o.. abrasion. This weight divided by 5 gives the percentage of wear."

SUMMARY FINE AGGREGATE

In order to secure suitable qualities, minimum requirements determined from the above tests should be definitely specified. For asphaltic-concrete sand requirements see page 494.

The following specifications in regard to cement-concrete sand are now being used by highway departments in several of the states:

"Sand for use in Portland-cement-concrete roads shall be of the following gradation: 100% shall pass a ¼" screen, not more than 20% shall pass a 50-mesh sieve, and not more than 6% shall pass a 100-mesh sieve. Sand may be rejected for this class if it contains more than 5% of loam and silt. Mortar in the proportion of 1 part of cement to 3 parts of the sand shall develop a compressive or tensile strength of a similar mortar of the same age, composed of the same cement and standard Ottawa sand.

"Sand for use in foundations, culverts, retaining walls, etc. shall not contain more than 8% of loam and silt. Mortar in the proportion of 1 part of cement to 3 parts of the sand when tested shall develop a compressive or tensile strength of at least 80% of the strength of a similar mortar of the same age, composed of the cement and standard Ottawa sand.

"Screenings, if substituted wholly or in part for the above sand, should meet the following requirements:

"They shall be free from dust coating or other dirt. One hundred per cent shall pass a ¼" screen and not more than 6%, shall pass a No. 100 sieve. Mortar in the proportions of 3 parts of the screenings or mixed screenings

[1] AGG'S "Construction of Roads and Pavements."

and sand with 1 part of cement shall develop a strength equal to a sand for which it is to be substituted.

"The best and safest way in the selection of a concrete sand is to have a fair representative sample from the deposit listed. After this is found to meet the requirements, it is necessary to have constant and careful field inspections and tests made as the deposit is worked.

"The use of screenings is not advisable on any concrete work, except where a good grade of sand is not available. When used, the product must be inspected constantly and tested, as it is likely to vary to a considerable degree. Screenings from the softer limestones should not be used, as the material is apt to "ball" in the mixer.

"Sand used for grout in brick and stone-block pavements must be fine enough to insure it getting between the joints of the block, but an excessively fine sand should be avoided, as it weakens the grout. Some states and many municipalities require the grout sand to pass a No. 20 sieve and not more than 30% pass a No. 100 sieve. Such sand should not contain more than 5% of loam and silt."

9. COARSE AGGREGATE

Coarse aggregate for use in structural concrete should be of hard, durable stone gravel or blast-furnace slag (see table of tests) free from coating of any kind. For use in concrete pavement, stone and gravel should be hard, tough, and absolutely clean. For use in culverts, retaining walls, etc., stone, gravel, or slag should be of sound, unweathered material, clean and free from coating. It should not contain more than 10% of soft stone or shale. Gravel containing a large percentage of thin, flat stone should not be used. Detail test methods follow.

For reinfored concrete the size of the stone is usually ½ to 1" in order to facilitate the compacting of the concrete between the reinforcing bars or mesh. For plain concrete a mixed size is used, ranging from ½ to 3½"; a scientifically graded stone reduces the amount of mortar required, but the structures in road work are so small that it does not pay to attempt to reduce the voids in this manner and the size that is available is used, varying the proportions of mortar to get a dense product. For extensive concrete pavement of the first class, graded sizes are feasible.

The use of slag in concrete is still a debatable matter but if proved to be reasonable will add materially to the source of concrete materials. All the indications and service tests seem to indicate that it is suitable for paving bases, but that its use for concrete roads is to be avoided unless other suitable materials are prohibitive in cost.

STANDARD METHOD FOR MAKING A MECHANICAL ANALYSIS OF BROKEN STONE, BROKEN SLAG, OR GRAVEL PEBBLES[1]

"The method shall consist of drying at not over 110°C. (230°F.) to a constant weight a sample weighing in pounds six times the diameter in inches of the largest holes required; passing the sample through such of the following size screens having circular opening as are required or called for by the specifications, screens to be used in the order named: 8.89 cm. (3½"), 7.62 cm. (3"), 6.35 cm. (2½"), 5.08 cm. (2"), 3.81 cm. (1½"), 3.18 cm. (1¼"), 2.54 cm. (1"), 1.90 cm. (¾"), 1.27 cm. (½"), and 0.64 cm. (¼"); determining the percentage by weight retained on each screen; and recording the mechanical analysis in the following manner:

[1] American Society For Testing Materials, Standard Method, Serial Designation: D18-16, slightly modified. AGG's "Construction of Roads and Pavements."

Passing 0.64-cm. (¼″) screen	...... %
Passing 1.27-cm. (½″) and retained on a 0.64-cm. (¼″) screen	...... %
Passing 1.90-cm. (¾″) and retained on a 1.27-cm. (½″) screen	...... %
Passing 2.54-cm. (1″) and retained on a 1.90-cm. (¾″) screen	...... %
	100.00%"

Weight per Cubic Foot and Void Tests on Coarse Aggregate.—The weight per cubic foot of coarse aggregate shall be determined as follows:

A cylindrical measure of at least 14 cu. ft. capacity with inside diameter approximately equal to inside height, or a box approximately cubical in shape and of not less than ½-cu. ft. capacity, should be used.

Ordinarily, the determination should be made on aggregate in air-dry condition. When the aggregate contains an appreciable amount of moisture, the percentage of water by weight should be determined and recorded.

About ¼ of the total amount of aggregate necessary to fill the measure is first introduced in such manner as to avoid separation of sizes. This material is then shaken down by rocking the measure from side to side until no further settlement takes place. The process is repeated until the measure has been filled to overflowing, after which it is struck off level with the top with a straightedge and weighed.

The percentage of voids in the aggregate may be determined from the weight per cubic foot and specific gravity in the usual manner.

Method of Determining the Weight of Fine Aggregate per Cubic Foot.—For tests on fine aggregate use a cylindrical metal measure having inside diameter equal to inside depth. A measure of capacity of ⅕ to ½ cu. ft. is suggested, but a measure as small as $\frac{1}{20}$-cu. ft. capacity may be used.

Ordinarily, the weight per cubic foot should be determined on air-dry material. When the aggregate contains an appreciable amount of moisture the percentage of water by weight should be determined and recorded.

Fill the measure one-third full, puddle with 25 to 30 strokes from a ½″ round steel bar 20″ long, pointed at the lower end. Continue filling and puddling in like manner until the measure is full, then strike off the top by a rolling motion with the bar. Determine the weight of the contents of the measure and calculate the weight in pounds per cubic foot.

Specific Gravity and Absorption Tests on Stone or Other Coarse Materials.—The apparent specific gravity is obtained by weighing the water displaced by a sample of the material weighing approximately 1000 g. broken into pieces about 1¼″ in diameter. A special type of vessel is used. It consists of a galvanized-iron cylinder closed at one end, and measuring 5″ in diameter by 8″ high. A brass spout ½″ in diameter is soldered into the side of the cylinder 6″ from the bottom. The spout is inclined at an angle of 2 with the horizontal and is 2½″ long. A notch is filed across its lower end, as shown, to stop the drip from the displaced water. In determining the specific gravity, the dried and cooled sample is weighed to the nearest 0.5 g. and immersed in water for 24 hr.

The pieces are then individually surface dried with a towel, the sample reweighed and immediately placed in the cylinder, which has been filled previously to overflowing with water at room temperature.

The weight of water displaced by the sample is used to calculate its apparent specific gravity. The difference between the original weight of the sample and its weight after 24 hr. is used to determine the absorption.

Abrasion Tests for Gravel.—The aggregate is screened first through screens having circular openings 2, 1½, 1, and ½" in diameter. The sizes used for this test are divided equally between those passing the 2" and retained on a 1½" screen, passing a 1½" screen and retained on a 1" screen, passing a 1" screen and retained on a ¾" screen, passing a ¾" screen and retained on a ½" screen. The material of these sizes is washed and dried. The following weights of the dried stone are then taken: 1250 g. of the size passing the 2" screen and retained on the 1½" screen, 1250 g. of the size passing the 1½" and retained on the 1" screen, 1250 g. passing the 1" screen and retained on the ¾" screen, and 1250 g. passing the ¾" screen and retained on the ½" screen. This material is placed in the cast-iron cylinder of the Deval machine, as specified for the standard abrasion test on stone. Six cast-iron spheres 1.875" in diameter and weighing approximately 0.95 lb. (0.45 kg.) each are placed in the cylinder as an abrasive charge. The spheres are the same as those used in the standard paving-brick rattler test.

After the cast-iron spheres have been placed in the cylinder the lid is bolted on and the cylinder mounted in the frame of the Deval machine. The duration of the test and the rate of rotation are the same as specified for the standard test for stone, namely, 10,000 revolutions at the rate of 30 to 33 r.p.m. At the completion of the test the material is taken out and screened through a 16-mesh sieve. The material retained upon the sieve is washed and dried and the per cent loss by abrasion of the material passing the 16-mesh sieve calculated.

When the materials have a specific gravity below 2.20, a total weight of 4000 g. made up of the four groups of sizes described above, instead of 5000 g., shall be used in the abrasion test.

Proposed Test for Percentage of Shale in Gravel.—It is suggested that, for the separation of shale and other light unsatisfatory pieces from concrete aggregate, a solution of zinc chloride ($ZnCl_2$) or some other satisfactory liquid having a specific gravity of approximately 1.95 be used. A sample of the pebbles should be first dried to constant weight at not over 110°C., and then placed in a container of suitable size partially filled with the solution. Agitate for 5 min., skim off the lighter materials, and then pour the solution through a sieve which will retain the pebbles. Repeat the operation until the entire sample has been separated. Dry to constant weight, measure the volume of retained material, and compute the percentage by volume of shale or other soft material.

STANDARD METHODS OF MAKING AND STORING SPECIMENS OF CONCRETE IN THE FIELD. SERIAL DESIGNATION: C31-21

"These methods are issued under the fixed designation C31; the final number indicates the year of original adoption as standard, or, in the case of revision, the year of last revision. Proposed as tentative. 1920; adopted, 1921, American Society for Testing Materials.

"1. **Scope.**—The methods herein specified apply to molding and storing of test specimens of concrete sampled from concrete being used in construction.

"2. **Size and Shape of Specimen.**—The test specimens shall be cylindrical in form with the length twice the diameter. In general, a mold whose diameter is not less than four times the diameter of the largest-size aggregate shall be used. (The sizes most commonly used are 6 by 12″ and 8 by 16″ cylinders.)

"3. **Molds.**—*a.* The molds shall be cylindrical in form, made of non-absorbent material, and shall be substantial enough to hold their form during the molding of the test specimens. They shall not vary in diameter more than 1/16″ in any direction, nor shall they vary in height more than 1/16″ from the height required. They shall be substantially water-tight so that there will be no leakage of water from the test specimen during molding.

"*b.* Each mold shall be provided with a base plate having a plane surface and made of non-absorbent material. This plate shall be large enough in diameter to support the form properly without leakage. Plate glass or planed metal are satisfactory for this purpose. A similar plate should be provided for covering the top surface of the test specimen after being molded.

"*c.* Suggestions for suitable forms are shown in accompanying figures.

"4. **Sampling of Concrete.**—*a.* Concrete for the test specimens shall be taken immediately after it has been placed in the work. All the concrete for each sample shall be taken from one place. A sufficient number of samples—each large enough to make one test specimen—shall be taken at different points so that the test specimens made from them will give a fair average of the concrete placed in that portion of the structure selected for tests. The location from which each sample is taken shall be noted clearly for future reference.

"*b.* In securing samples, the concrete shall be taken from the mass by a shovel or similar implement and placed in a large pail or other receptacle, for transporting to the point of molding. Care shall be taken to see that each test specimen represents the total mixture of the concrete at that place. Different samples shall not be mixed together but each sample shall make one specimen.

"5. **Molding the Specimens.**—*a.* The pails or other receptacles containing the samples of concrete shall be taken as quickly as possible to the place selected for molding test specimens. To offset segregation of the concrete occurring during transportation, each sample shall be dumped into a non-absorbent water-tight receptacle and, after slight stirring, immediately placed in the mold.

"*b.* The test specimens shall be molded by placing the concrete in the form in layers approximately 4″ in thickness. Each layer shall be puddled with 25 to 30 strokes with a 5/8 to 3/4″ bar about 2′ long, tapered slightly at the lower end. After puddling the top layer, the surface concrete shall be struck off with a trowel and covered with the top cover plate, which will later be used in capping the test specimens."

"6. **Capping Specimens.**—Two to four hours after molding, the test specimens shall be capped with a thin layer of stiff neat cement paste in order that the cylinder may present a smooth end for testing. The cap can best be formed by means of a piece of plate glass 1/4″ thick and of a diameter 2 or 3″ larger than that of the mold. This plate is worked on the fresh cement paste until it rests on top of the form. The cement for capping should be mixed to a stiff paste some time before it is to be used in order to avoid the tendency of the cap to shrink. Adhesion of the concrete to the top and bottom plates can be avoided by oiling the plates or by inserting a sheet of paraffined tissue paper.

"7. **Removal of Specimens from Forms.**—At the end of 48 hr. the test specimens shall be removed from the molds and buried in damp sand except in case the molds shown in Fig. 3 are used; in this case test specimens may be buried in damp sand without removal of the mold, thus permitting shipping of the test specimens in the molds.

"8. **Storage of Specimens.**—*a.* The test specimens shall remain buried in damp sand until 10 days prior to date of test. They shall then be well packed in damp sand or wet shavings and shipped to the testing laboratory, where they shall be stored either in a moist room or in damp sand until the date of test.

"*b*. Should a 7-day test be required, the test specimens shall remain at the works as long as possible to harden and then shall be shipped so as to arrive at the laboratory in time to make the test on the required date."

TENTATIVE METHOD OF TEST FOR CONSISTENCY OF PORTLAND-CEMENT CONCRETE. SERIAL DESIGNATION: D138-25T

"This is a *Tentative Standard* only, published for the purpose of eliciting criticism and suggestions. It is not a Standard of the Society and until its adoption as Standard it is subject to revision. Issued, 1922, American Society for Testing Materials.

"1. **Scope.**—This test covers the method to be used both in the laboratory and in the field for determining consistency of concrete[1] to be used for concrete pavements or for concrete base for pavements.

"2. **Apparatus.**—The test specimen shall be formed in a mold of No. 16 gage galvanized metal in the form of the lateral surface of the frustrum of a cone with the base 8" in diameter, the upper surface 4" in diameter, and the altitude 12". The base and the top shall be open and parallel to each other and at right angles to the axis of the cone. The mold shall be provided with foot pieces and handles as shown in accompanying figure.

"3. **Sample.**—When the test is made at the mixer, the sample shall be taken from the pile of concrete immediately after the entire batch has been discharged. When testing concrete that has been hauled from a central mixing plant, the sample shall be taken from the concrete immediately after it has been dumped on the subgrade.

"4. **Procedure.**—The mold shall be placed on a flat, non-absorbent surface, such as a smooth plank or a slab of concrete, and the operator shall hold the form firmly in place, while it is being filled, by standing on the foot pieces. The mold shall be filled to about one-fourth of its height with the concrete, which shall then be puddled, using 20 to 30 strokes of a ½" rod pointed at the lower end. The filling shall be completed in successive layers similar to the first and the top struck off so that the mold is exactly filled. The mold shall then be removed by being raised vertically, *immediately* after being filled. The molded concrete shall then be allowed to subside until quiescent and the height of the specimen measured.

"5. **Slump.**—The consistency shall be recorded in terms of inches of subsidence of the specimen during the test, which shall be known as the slump.

Slump = 12 − inches of height after subsidence."

Test for Clay in Sand Clay, Topsoil, and Semigravel.—Dry 500 g. of the material at a temperature below 350°F. (176.6°C.) to a constant weight. Gently pulverize to break down soft clods or masses, but not to grind or break hard material. Pass through a 10-mesh sieve, weigh the coarse residue, and record as "coarse material." Use the material passing through the 10-mesh sieve as the starting point of a percentage analysis as follows:

Weigh out two samples of 50 g. of this material for duplicate analysis. Place each in a tared wide-mouth bottle (5- to 6-cm. diameter and about 12 to 15 cm. high). Add about 5 cc. of dilute ammonia water and about 200 cc. of water. Close with a cork or glass stopper and shake thoroughly for 20 min. Allow the sample to settle 8 min. and decant carefully or siphon off the supernatant liquid to a depth of 8 cm. below the surface of the liquid. (The depth of the liquid in the bottle should be sufficient to leave about 4 cm. below the point of siphoning.) Fill the bottle again with water, shake for 3 min., allow settlement, and siphon off as before.

[1] This test is not considered applicable when there is a considerable amount of coarse aggregate over 2" in size in the concrete. The committee is now working on a method suitable for determining the consistency of concrete using aggregate over 2" in size.

Repeat the process until the supernatant liquid is clear. Be careful to wash the stopper and neck of the bottle free from coarse material before decanting.

Dry bottle and washed material to constant weight at between 100°C. (212°F.) and 110°C. (230°F.), weigh and determine net weight of washed material.

$$\frac{\text{Original weight} - \text{washed weight}}{\text{Original weight}} \times 100 = \text{per cent clay and silt.}$$

As a check the washings drawn off shall be collected and evaporated to dryness for direct recovery of the fine sediment classed as clay and silt.

$$\frac{\text{Weight of residue}}{\text{Original weight}} \times 100 = \text{per cent clay.}$$

The determinations on the two samples shall check within 1% to be acceptable.

Tests for Semigravel, Topsoil and Sand Clay.—Wash the contents of the bottle cleanly into a porcelain evaporating dish and carry to dryness on a water bath. The dried residue should be carefully scraped from the dish and passed through a nest of 20-, 60-, 100-, and 200-mesh sieves. The residue retained on each sieve is weighed and recorded as sand of the respective sizes. Their sum constitutes the total "sand." The residue passing the 200-mesh sieve and caught in the pan is weighed and recorded as "silt." Duplicate samples should check within 1%.

a. The coarse material should be examined for hardness and with the magnifying glass to identify its character as quartz, hard-iron compounds, feldspar, schistose material, or indurated clay. Hard quartz or iron gravels are valuable in themselves and as indicating the quality of the finer aggregate. Feldspar, mica, and clay nodules are worthless and indicate that the accompanying soil is poor for road building.

b. The sands should be examined with the magnifying glass for identification as quartz and for the presence of mica scales or feldspar needles. If mica or feldspar is present in appreciable amounts the sample should be rejected.

c. When the clay is recovered by evaporation, it can be examined for tenacity by cementing together two glass plates, each 1" wide, set at right angles, with a layer of clay whose thickness is fixed by a fine bent wire laid between the plates. The moist clay covers the wire on one plate, and the other plate is squeezed down tightly on the wire. After drying, the one plate held firmly against cleats, wire slings are run symmetrically from the ends of the upper plate to one arm of a beam balance, and the tension necessary to separate the plates is given by shot or weights in the other pan of the balance. This test is tedious and is of service chiefly on low-grade samples which are of doubtful efficiency, but which represent the only available material for local construction.

d. Approximate test for tenacity of mixture can be made as follows:

Make cylinders from the material passing the 10-mesh sieve, 25 mm. The material is worked into a stiff mud and molded under 132 kg. per square centimeter pressure. Dry thoroughly at 100°C. (212°F.) and break by the small Page impact machine for testing cementing value, using a 1-kg. hammer and 1-cm. drop. Record the number of strokes as the relative measure of tenacity.

Usually the plastic character and adhesiveness of a good road soil can be judged by the feeling of the mud made from this material, its adherence to the hands, and its strength under light pulling.

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