ROORKEE TREATISE ON CIVIL ENGINEERING

SECTION VII

BRIDGES

NINTH EDITION

(Reprint)

BY

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INCORPORATED WITH

A CHAPTER

ON

STEEL BRIDGES

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ALLAHABAD:

PRINTED BY THE SUPERINTENDENT, PRINTING AND STATIONERY, UNITED PROVINCES, INDIA

1947

On sale at the Book Depot, Govt. Photo Litho Press, Roorkee

PREFACE

THE last revised edition of this Manual was published in 1897. Since then Bridge Engineering has made considerable progress and it has become necessary to rewrite the Manual revising those parts which need bringing up to date and adding fresh matter where it is absolutely necessary to meet present requirements.

The introductory chapter and the chapter on Reinforced Concrete Bridges are entirely new, while the chapters on Foundations and Iron or Steel Bridges have been rewritten to accord with modern practice. The chapter on Temporary Bridges has been taken largely from some very useful notes on the subject recently prepared for the College by Major A. H. Cunningham, R.E.

In writing the Manual, the Author has endeavoured to confine himself to practical details of construction and to avoid elaborate mathematical calculations as far as possible, but, in a technical subject of this kind, it is not possible to make all explanations clear without some reference in places to the theories on which practice is based. Where the calculations involved were of a lengthy and complex nature as in the case of iron, reinforced concrete, and arch bridges, he has put them in an Appendix to simplify the text as far as possible.

Many of the recent publications on the subject of bridge construction have been consulted in collecting up-to-date information for the Manual but the Author is chiefly indebted to the following books and papers :

1. Notes on temporary bridges by Major A. H. Cunningham, R.E.
2. Minutes of proceedings of the Institution of Civil Engineers.
3. "Civil Engineering" by Vernon Harcourt. Published by Messrs. Longmans, Green and Co., London.
4. "Manual of Reinforced Concrete", by Marsh and Dunn. Published by Messrs. Constable and Co., London.
5. Reinforced Concrete by Buel and Hill. Published by the Engineering Publishing Co., New York.
6. Notes on Reinforced Concrete by Captain J. Fleming, R.E. Chatham, 1910.

C. E. V. G.

10th September, 1915.

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CHAPTER I

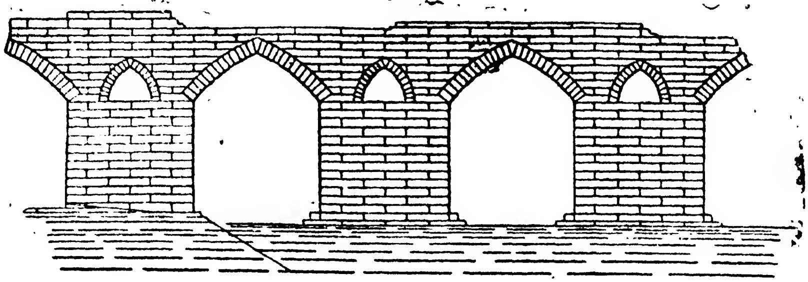
INTRODUCTION

Origin and development of bridge building—Existing ruins and ancient records indicate that bridges have been used since the dawn of history. In pre-historic times, primitive man crossed rivers on rafts or rough boats, and smaller streams by rude structures of logs or stone slabs thrown across them or on stepping stones, but the growth of civilisation and commerce created a need for more substantial and permanent crossings. The human race is supposed to have originated in Assyria, Babylonia and Egypt, and the first traces of bridges have been found in these countries. The art of bridge construction has been evolved gradually with the building of cities and the development of civilisation from age to age. The materials used in ancient times seem to have been much the same as now, the difference in construction being chiefly in the forms employed in putting the materials together. The bridges of a very early date were more or less of a temporary nature owing in those days to the constant liability to war during which bridges offered facilities to the invader as they now do to commerce. Many of the ancient bridges best known in history were built during military campaigns for the transportation of armies, and they are described as some of the most important achievements of those who conducted the campaigns.

2. **Arched bridges**—The most common form of bridge is the arched type. The date when arches were first introduced is uncertain but they have been found in recent excavations at the supposed sites of Nineveh and Babylon which were destroyed in the seventh century B. C. These arches covered underground structures which are supposed to have been sewers. In the ruins of Khorsabad, 15 miles from Nineveh, semi-circular voussoir arches of 13 feet span have been found over gateways in the City wall which date back to at least 720 B. C. These are some of the earliest known traces of the true arch, but false arches over doors and gateways have been discovered in even more ancient ruins. Most of the early forms were really corbels, bracketted out from the supporting piers and meeting at the span centre, exerting only vertical reactions.

In Persia, the oldest existing bridge is over the river Diz in Kurdistan. It is 1,250 feet long. It was built 350 B. C. and is still in a fair state of preservation It consists of 20 pointed arches, each of 25 feet span, between piers 29 feet thick. A sketch is given below, Fig. 1. The large pointed arches and the small ones above the piers are characteristic of the architecture of those days.

Fig. 1.



Masonry arches are known to have existed in China long before the Christian era. They were used for carrying the well known "Great wall of China" over streams and rivers. There are no records of ancient bridges in this country, but the Chinese are believed to have been among the earliest bridge builders and contemporary with the Babylonians. They used both slab and arch construction.

The bridges of ancient Greece, 500 B. C. and earlier, were built with heavy piers and abutments of stone spanned by wooden beams or stone lintels dowelled together. The Assos bridge, a good type of ancient Grecian bridges, had 17 piers, diamond shaped in plan and 10 feet apart at the nearest angle, with stone lintels 20 inches thick and 24 inches wide, dowelled together. In later years, Grecian bridges were built with the upper stone courses projecting, one beyond another, to meet in the centre in the form of a false triangular arch. The semi-circular arch used at a later date was developed from the earlier false arch.

The old Roman bridges have shown the greatest permanence. There are still about 20 in existence which are built at or before the commencement of the Christian era. For 2,000 years or more they have resisted the destructive effects of floods, earthquakes, and wars, and many of them are still in a good state of preservation. Though built without much theoretical knowledge of their stresses, they are well designed and constructed. The arch stones were fitted so carefully that they appear to have been ground together. No mortar was used in the joints but the stones were bound together with iron clamps. Up to the 6th and 7th centuries B. C. the piers of Roman bridges used to be corbelled out to meet at the centre, but after that the semi-circular arch began to appear and all Roman arches were of that form. The span of the old Roman bridges rarely exceeded 70 to 80 feet and the piers were very thick, often one-third of the span. They generally had an uneven number of openings with the spans decreasing in length from the centre to the ends.

3. **Wooden bridges**—Herodotus, "the father of history" has given us the earliest record of a wooden bridge. This was built 783 B. C. over the river Euphrates in Babylon. It had stone piers connected with wooden platforms which were removed at night to prevent thieves from entering the city. It was 600 feet long and 35 feet wide. Another historian says the bridge was covered by a roof and it rivalled in beauty and magnificence any other structure in Babylon.

4. **Floating bridges**—Floating bridges, with casks as floats, were used for military purposes in very early times. They are mentioned by Homer as common in his time, about 800 B. C., and they are referred to in the writings of Herodotus and Xenophon. The earliest bridge of this kind of which details are recorded was built by Cyrus, King of the Persians, in the year 536 B. C. He used inflated skins as floats.

5. **Suspension bridges**—The suspension bridge is one of the earliest types known. It was used in remote ages in China, Japan, India, Tibet, Mexico and South America. In all early forms, the road platform was carried directly on the cables which were usually twisted vine or straps of hide stretched tightly to remove sag. The cable ends were fastened to trees or other fixed objects on shore. The first bridge of this kind of which there is any record in history was built 65 B. C. by order of the Emperor Ming in the province of Yunnan in China. It was 330 feet long with a plank floor resting on two chains. This type of bridge has not developed as rapidly as other forms owing chiefly to its lack of rigidity. A more primitive type, still in use in remote parts of the hills in India, consists of a single rope with a basket suspended from it. The latter is drawn backwards and forwards on the rope by a smaller rope or cord.

6. **Iron and steel bridges**—Iron and steel in the form of arched ribs, girders and cantilevers were not used as materials of bridge construction till the middle of the 18th century. The first cast iron arch bridge was built in England over the Severn river at Coalbrookdale in 1776 and is still in good condition. It has a span of 100 feet with a rise of 45 feet. There are five ribs, and each rib has 3 concentric rings joined together by radial pieces. This was the first bridge, composed entirely of iron, built in any country. The first useful wrought iron girder

bridge was built by George Smart in 1824 on the Dublin Drugheda Railway. It consisted of lattice trusses of 84 feet span composed of vertical framing and diagonals inclined at an angle of 80° to the vertical.

7. **Reinforced concrete bridges**—These are of very recent origin. Iron and copper clamps have been used for some time on the extrados of an arch to prevent joints opening up, but the advantage derived from such clamping was not very great as it did not provide for the moment producing tension on the intrados, and failure could take place by the opening up of the joints on the lower side. The general application of reinforced concrete in engineering and architectural works dates from the introduction of what is now known as the Monier system by Jean Monier of Paris in 1876. At first Monier only used one system of netting on the lower side, but this offered no resistence to the moment producing tension in places in the extrados and, as in the case of the old clamped voussoir arches, the metal reinforcement did not furnish any considerable increase in strength. Monier arches were afterwards built, as they now always are, with two nettings, one on the upper and one on the lower side. Further developments of this method of construction will be found in Chapter VI.*

8. **Various types of bridges**—Bridges may be either floating or fixed, temporary or permanent, and may be made of rope, wood, brick, stone, concrete and reinforced concrete, iron or steel. In design they may be classified as temporary bridges, wooden bridges, suspension bridges, arched bridges of masonry, brickwork, or concrete, reinforced concrete bridges, iron and steel bridges. In the following chapters they have been described in order according to this classification.

*Students who wish to make a further study of the gradual development of the art of bridge building should read "History of Bridge Engineering," by H. G. Tyrrell, C.E.

CHAPTER II

SITE, WATERWAY AND DESIGN

9. **Site**—When the place where a bridge is to be erected is not determined by the position of a road which it is inexpedient to alter the site which offers the most security for the foundations, the straightest reach of river up and down stream at right angles to the crossing, and the greatest economy in construction is to be preferred. If rocky foundations can be had, they will save much expense and anxiety; if not, any place where the river runs between high permanent banks will be good, as the course of the river will not be liable to alteration and the amount of water to be passed is easily known. In the plains of Northern India, however, neither of these conditions is often met with. From the flatness of the country and the vast quantity of silt brought down by the waters of an Indian river during the heavy freshes caused by the melting of the snow in the hills and the periodical falls of rain, the bed levels and the course of the stream are perpetually being altered. No one who has not watched one of these large rivers for some years would believe the extraordinary changes that occur in quick succession. Take for instance the Jumna at Okhla, below Delhi; the river discharged 140,000 cubic feet per second in the floods of 1871, and only 3,800 cubic feet per second in the winter of 1871-72. Even in cases where the bed and banks are comparatively stable, great variations in flood volume are caused by exceptional falls of rain; thus, near Kasganj on the 17th July, 1885, a flood exceeding 100,000 cubic feet per second passed down the Kali Nadi, the maximum known flood up to 1870, being only 26,382 cubic feet per second. Again consider the case of the Indus above its junction with the other Punjab rivers; its discharge in winter is 13,000 cubic feet per second, in summer at least 120,000; its proper banks at this place are three to four miles apart; its cold weather channel in one year may be 1,000 feet across, and in the next year in the same place there will be three channels instead of one, separated by islands of sand; the ferry has often to be shifted two miles or more, up or down stream in two following years; in four years it cut away $1\frac{1}{2}$ miles of bank, measured perpendiculary, and threatened destruction to a large town, and the next year the deep stream shot off to the opposite side three miles from the town.

10. It is evidently impossible, except at an enormous expense, to bridge the whole of such a stream as has been described, nor indeed

is it necessary or advisable. What must be done is to provide waterway for the largest amount of water that can ever be expected to pass, and at such a velocity as shall not be dangerous, and then to connect the bridge with the high ground on each side by massive embankments carried across the valley of the river, culverts of suitable waterway being provided, if absolutely necessary, where these embankments cross natural side channels or spills from the river.* For convenience sake, the bridge might be built in the dry bed near one of the banks and the water turned through it when it is ready, it being understood that the natural or artificial reach thus bridged should be as long and as nearly at right angles to the bridge as possible. It is also evident that the river, especially if liable to scour, should have as few assailable points presented to it as possible, i.e., that the piers should be few in number; which, of course, implies large spans. Hence is seen the great advantage in the employment of iron girder bridges for large rivers. We can get in the foundations gradually during successive cold seasons; then proceed with the piers, and finally with the superstructure, without having recourse to numerous centerings for turning a large number of arches at once, which are often surprised by the river, carried away, and the stability even of the finished ones endangered by all not being completed. In fact, the difficulty of constructing arches of large span within a limited time in masonry or concrete (unless reinforced) is often practically insuperable, and, with small arches, great obstruction is caused by numerous piers, so that by the time a large bridge has been finished it has occasionally happened that the river has taken new course altogether, leaving the bridge high, and dry, and the water cannot be brought through it except at a ruinous expense.

11. With large spans there is always the possibility of the river carrying away the road embankments, especially when the earth is new, and they must be watched very carefully at every point across the whole valley, often many miles in extent. Their thickness must be sufficient to withstand the height of water likely to be dammed up by them in the highest floods, they must be carefully made of rammed earth with a long flat slope on the water-side, and they must be well turfed or pitched where such protection is necessary. Moreover, the river itself must be well watched for some miles upstream, and every effort made to keep the stream straight and uniform in its new channel. Little can

*See para. 16 (6) as to the objections to bridging these side channels. They should be closed as a rule, but if for any special reason they must be kept open they should be controlled by sluice gates.

be done in this way when the river is in flood, but in the cold season much may be effected with comparatively small means if judiciously applied.

12. **River-training**—Though this operation strictly comes under the head "Improvement of Rivers" which is dealt with generally in the Irrigation section of these manuals, there are certain points of vital importance in bridge building which must be fully explained in this Manual. The guiding principle to be kept in view is that to turn a stream *by direct opposition* is almost impossible, while to *lead* it is comparatively easy if we go to work in the right way. Any subsidiary channel down which the main stream shows a tendency to take its course should be dammed across at the head; for this purpose, sand dug from the bed of the river, may be used, strengthened, if necessary, by piling; or where the velocity is great, tree spurs, aligned so as to cause a diversion of the current and a silt deposit in the side channel, may be fixed with ropes and anchors. Meanwhile, the set of the stream above the bridge should be carefully regulated and any tendency to cut away its straight banks checked. For this purpose, fixed spurs or floating breakwaters are the most practicable means for small streams, and "Bell's Bunds" for large rivers with wide sandy beds

13. **Spurs** may be formed of a double line of piling filled in with brushwood and carried out from the shore either perpendicularly or at an angle to the current. Perpendicular spurs are useful against weak currents. For high velocities the spurs should make an angle with the bank of not more than 30°. The force of the water is checked where it impinges on the spurs, a new direction is given to the current, and a considerable deposit of silt takes place both above and below the spur. It is calculated that each spur will protect a length of shore five times its perpendicular distance from the bank down-stream, and three times upstreams, with moderate velocities.

Where the water becomes too deep and rapid to continue the spurs, floating rafts of wood, trees, or fascines may be fixed to their ends, and kept in line by guy ropes fixed on shore, or by anchors. These floating *breakwaters* may also be employed alone; they change the set of a current so long as they are in their places, but cause a much smaller deposit of silt, and are, therefore, not so effectual a remedy as the fixed spurs.

Permanent stone or kankar groynes, with cross spurs and revetments, have been extensively used for the training of rivers near large bridges and other river works. The latest development of protective embankments for controlling the flow of large shifting rivers with wide sandy beds is the Bell's Bund system described below. Before this system of protection and training was adopted, great difficulty was experienced in securing uninterrupted communication over such rivers in the Punjab. In heavy floods, the piers or embanked approaches were often destroyed when the uncontrolled current directed its concentrated force on one or two points of the bridge, or in a direction not perpendicular to the bridge face.

The following reprint of Government of India Technical paper No. 2B gives a full description of these "bunds" by Mr. J. R. Bell who first devised the system and after whom they are called.

"In the Punjab Rivers—which erode their banks and scour their beds deeply on the outer edges of the erosive bends—our practice is to retain the stream within a limited length of the bridge by protective bunds faced and aproned with rough stone pitching. In cases where there is no solid ground on which the heads of these flank bunds can rest, we make their length upstream at least equal to the length of the bridge itself, and extend both of them downstream beyond the abutments to a distance of at least one-fourth the bridge length.

The alignment that is thought best on abstract considerations is sketched in *Fig.* 2. In the absence of natural heads those at AA are strengthened by very large mounds of stone in cases as much as 300,000 cubic feet per head. The extent to which the river should be threated between A and A depends on the number of piers in the bridge as these so obstract and sub-divide the channel that a very much narrower width at AA gives a much larger effective channel than that afforded by the bridge. The object of *vena contracta* on plan is to centre the river and make it fan out equally in all the spans. As a rule the conditions of the site do not admit of using the *vena contracta* ground plan, and it is found that where the bunds diverge on the upstream side there is a proportionate tendency for an island to form in the middle of the bridge which splits the deep channel towards the abutments. The downstream tails of the bunds as BB are necessary to counteract the eddy that tends to undermine the ground below either abutment.

The cross section of bund now in vogue is shown in *Fig.* 3 and the main factor in determining its proportions is the normal deep scour of an erosive bend referred to as "s" in the following portion of this note. Factor is not always easy to ascertain and should be carefully discriminated from the enormous depths attained in purely alluvial strata by eddies. For example, in the Chenab and Sutlej, eddy scours of 60 and even 70 feet below High Flood Level are known to occur, while 40 feet is the normal erosive scour (the S. of the diagram). Where rock is found overlaid by other than firm strata of probably considerable age, the rock is sure, sooner or later, to be scoured clean. At Sukkur the Indus cleans its rocky bed almost every year at 120 feet below High Flood Level. The dotted section shows the ultimate position of the apron when scour has engulfed it.

After deciding on the centre line of the bund (which should in all cases be at least 20 feet wide on top with slopes not steeper than 2 to 1 from three feet above High Flood Level down to the spring level at which water is encountered when the river is low) the apron pit is laid out with a width from toe of slope at spring level outwards $= \frac{5}{4}$ S. The bund is made wholly from the apron pit and if more earth is wanted, it is dug from the river side as borrow pits in rear of the bund are very objectionable and liable to induce "blows". Where the apron pit yields more earth than is absolutely requisite the width of bund is increased till they balance. The core of the bund should, *if* possible, be of fine sand and the slopes of good clay ; while the rear slope should be wattled and planted with willows, elephant grass, or other deep rooted vegetation as protection against the lap of wavelets that arise on the lake which forms in rear of the bund by spill or percolation.

In the Punjab this lake is purposely filled by a controllable sluice inlet that brings in silt to warp up the lake bed. This process is only effective when a high level outfall draws off the clean upper water and keeps up a steady influx of silt during flood time.

We now invariably lay the apron 4 feet thick, and hence its cross sectional area $= S \times 5$. The total amount of stone laid in at first $= S \times 8$ and the surplus or reserve $= S \times 3$ is stacked on the river slope of the bund at as steep an angle as it will stand, usually a little steeper than 1 to 1. The top of the bund carries a tramway by which the reserve can be transferred to any point where heavy scour threatens to engulf the reserve stone already in place. During the first three or 4 years of its existence we think it essential to renew the entire reserve when the river is low, and even to increase it when the indications point to our having underestimated the factor S.

Sharp-edged stone is best; and round or even cubical pieces are found wasteful and inefficient. The individual pieces should be of approximately one size and their weight ought to be the greater as the velocity of the stream increases. For 6 feet per second average velocity, stones averaging 1 cwt. suffice, if sharp-edged and of high specific gravity, to revet a subaqueous slope as far down as the scour extends. We attribute the success in point of stability and economy that has so far invariably attended this method as compared with that of stone faced spurs to the fact that the latter provoke and intensify *eddy scours*, while the former tends to eliminate eddies and to straighten out and so minimise the attacks of *bend scours*.

Fig 2

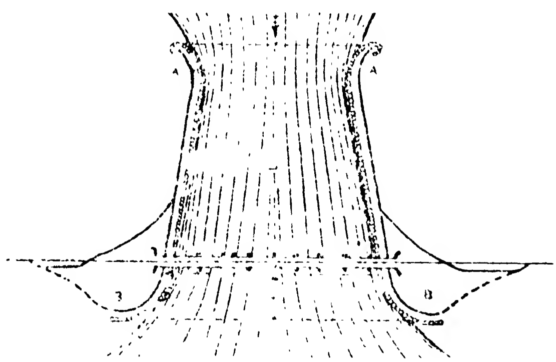
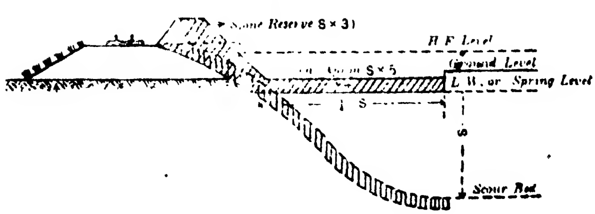


Fig 3.



15. Several years after these bunds were first introduced in the Punjab by Mr. Bell, Technical Paper No. 153 was written by Mr. Francis Spring, Chief Engineer, P. W. D., giving the results of the experience gained in different parts of India in constructing guide banks on this principle. This paper contains exhaustive information on the subject and should be studied carefully by all Engineers undertaking large bridges over wandering alluvial rivers. The most important points brought forward in the paper regarding the design of guide banks are summarised in the following notes for convenient reference. The student should defer reading these notes till he has read the rest of this chapter which follows the notes from paragraph 17 onwards.

1. To ascertain the actual length of bridge required between a pair of guide banks proceed as follows :—Find the maximum flood discharge at the selected crossing by one of the methods described further on in this chapter. Plot the cross section of the river at the crossing showing the highest flood line and on it draw a revised ideal section deeper all over than the existing one by 8, 10, 12, 14 or 16 feet according as the bed consist of very coarse, coarse, medium, fine or very fine sand. Mr. Spring's classification of sands is noted on the margin. Pick out the deepest and best part of this deepened section, and calculating the velocity for this section by Kutter's formula find out how much of it will suffice to carry the estimated flood discharge. The part so arrived at will be amply sufficient to carry the entire river, and the remainder may be blocked up.

Very coarse.—Quartz— fragments free from mica 40 % stopped by a 30-wire mesh and remainder by a 40 wire.

Medium.—Half quartz, half mica none stopped by a 30-wire mesh and none to go through a 100-wire mesh except a mere trace.

Very fine.—Micacheous silty sand, 90 % to pass through a 100 wire-mesh.

2. If practicable it is better that the guide banks should approach each other somewhat near their upper ends as suggested by Mr. Bell in the technical paper reproduced above, but the choice between parallelism, convergence and divergence must be dictated to some extent by the condition of the bed of the river for construction purposes during the working season, having re ard to the fact that the apron of the "bund" should be carefully laid on the dry bed or in shallow water and not in deep water if this can possibly be avoided. In any case the pair of guide banks ought if possible to be disposed symmetrically on either side of a straight line normal to the centre of the bridge.

3. The length of the upstream guide banks may be made equal to the length of the bridge. But due attention should be paid to the possibility of the river bending round about and behind one of the guide banks into the still water area at the back of it and eroding the main approach embankment. In specially wide *khadirs*, this feature may involve the use of much longer guide banks.

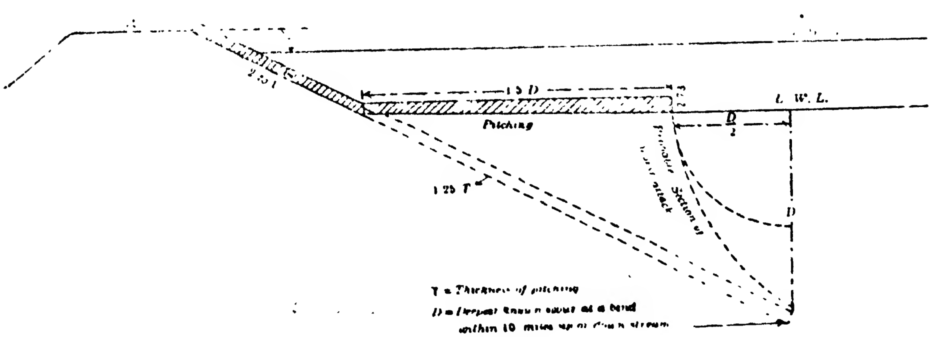
4. The length of the downstream part of the guide bank may be a tenth to a fifth of the length of the bridge according to the judgment of the Engineer as to the activity of the swirl or disturbance likely to be caused by the speading out of the water on leaving the bridge, for the swirl, if there is one, must be kept away far enough not to endanger the main approach embankments.

5. The radius of the downstream curved part of the guide bank may be something between 300 and 600 feet, and that of the upstream curved past between 500 and 1,000 feet according to the estimate that may be formed of the probable velocity of the

current pass it. The upstream curves should be curved well round to the back: fully 140°. It has been observed in rivers of the class under reference that water at the velocity of 8 to 10 feet per second, led fairly past a curv 600 to 800 feet radius, will pass smoothly along the perimeter without producing swirls. For lower velocities, the radius may be smaller.

6. It is very important that still water areas behind the guide banks should be carefully maintained, and minor bridges or large culverts should not be built in the approach enbankments of size sufficient to cause disturbing currents within this area. Not only may the existence of a current through such flood openings have some effect in drawing the main river bend towards them so far as to be dangerous to the rear of the guide bank or even the main bridge embankment itself, but it may cause a minor dengerous current along the comparatively unarmoured approach bank and produce a serious breach in it. If, for any reason, a flanking flood opening is unavoidable behind a guide bank it should be sluiced to control the flow through it, and the bed and banks on both sides should be heavily pitched to guard against scour. Though large flood openings near guide banks are objectionable for reasons above stated, quite small culverts about (2'×4') may prove very useful for a number of years to aid in the gradual silting up of the still water bay, for, naturally the greater the quantity of silt laden water that passes slowly over the bay the more silt will be deposited.

7. The guide bank section and its armour should be as follows:—The earthen part of the bank should be 20 feet to 30 feet wide on top with 2 to 1 slopes on both sides and a freeboard of at least 4 feet over the highest flood level. Borrow pits to be on the river side. If the banks are entirely of sand, then uncovered rear slopes should be coated with a foot or two good earth to encourage growth of vegetation and long grass. The thickness of pitching on the river slope will vary according to the flood velocity expected. In very coarse sand, the thickness will be from 16" to 28" for velocities ranging from 6·7 feet per second to 10·5 feet; in medium sand from 28" to 40", and in very fine sand from 40" to 52" for the same range of velcities. The pitching should be of rough angular stone or block kankar, and round stones should be avoided as far as possible. Towards the face the stones should weigh 60 to 120 lb., but the stone next the earth bank might be smaller. The thickness of pitching noted above may be reduced 6" all round on the slope if the stone is carefully graduated in size, laying small quarry refuse next the sand and the larger stone outside. The apron should be laid as shown in the sketch below which explainst itself



16. With reference to the foregoing, the following notes have been contributed by Mr. F. R. Bagley, M.I.C.E., late Chief Engineer, North-Western State Railway, who has had a very wide experience of bridge building in India :

1. The length of Bell's "Bands" above a bridge bears no relation practically to the length of the bridge. The main point to be considered in deciding what the length of the "bands" should be is the extent to which the river is likely to cut in at the back of the "bands" before it makes a short cut across the "bands" and staighten itself out. No Punjab river has been known to cut in more than 500 feet behind properly designed "bands" and allowing a margin of 500 feet a length of 1,000 feet on the upstream side is ample for such rivers.

2. Similarly there is no connection between the length of the bridge and the length of "band" required on the downstream side ; 800 feet is an ample length for the "band" below the bridge.

3. Curves at the terminations of the "bands" above the bridge do not make the current pass smoothly along the perimeter as stated by Mr. Spring, but deflect it at a tangent producing destructive eddies. Upstream heads should be square and parallel with the flood stream finishing off in long flat slopes, 1 to 7, and a heavy apron.

4. The thickness of the apron should vary from 7′ at the heads to about 3″ on each side of the bridge.

5. The guide banks should be laid out perpendicular to the bridge without any divergence or convergence.

6. The top of the guide banks should be 8 feet above high flood level as an afflux of 5 feet has sometimes been known to occur in the bay behind the bank.

7. A thickness of 18″ to 24″ is sufficient for the pitching of the water face above the apron. The pitching should be carefully hand-packed. All the individual pieces need not be of one size—small pieces to the extent of 25 per cent. are useful in filling interstices. Round pieces are not so good as angular, but, in places where angular stones are not readily available, boulders may be safely used in the lower parts If the upper layers are of sharp-edged stone. If nothing but boulders can be obtained at a reasonable cost the upper layers should be of round stone enclosed in stout wire netting.

8. With reference to paragraph 15 (1), the usual practice is to allow 15 feet of scour in Punjab rivers and take a velocity of about 10 feet a second through the waterway allowed.

17. When straightening the channel of a river to provide a fair run on to and through a bridge care must be taken not to shorten the course unduly. It is evident that the natural length of the channel of a river running in soil liable to erosion is determined by the velocity of the current, or more properly speaking by the silt-carrying capacity of the stream. Now this capacity is, to a great extent, influenced by the slope of the bed, and shortening the length of the channel will certainly increase the slope and the erosive power of the river, resulting either in charges of course in the river at a distance from the bridge, or in destructive action on the training works in the straightened reach.

This simple condition is frequently overlooked in designing training works. Its neglect is sometimes the cause of great scour under pier bridges and of heavy action above and below those provided with floors.

18. When rivers occupying broad flat valleys are liable to excessive floods at long intervals of time, and the cost for *road bridges* of full waterway is considered prohibitive, metalled *dips* or paved causeways at suitable points in the embanked approaches may be provided as a cheap and moderately efficient expedient. In such cases the permanent bridge should be given waterway sufficient for the normal floods, and the dips made large enough to pass the surplus due to the maximum discharge with a moderate velocity. The sites and dimensions of dips should be designed so as to prevent the formation of large side channels which might endanger the stability of the bridge, or the permanence of the main river channel. Dips on well foundations have been made in some places in the Punjab, even on branch railway lines, to keep down the cost of the lines as far as possible where the depth of water is small and floods occur but seldom.

19. The site of the bridge should be carefully watched even after the bridge has been built and the river has been taken fairly through it. A little care and attention before and during floods and the judicious employment of bunds and spurs will, in a short time, cause a silt deposit up to flood area level on both banks of the river. This will reduce the danger of future changes, strengthen the approaches, and improve the appearance and healthiness of the site.

WATERWAY

20. The Engineer's next step, after deciding where to cross, is to ascertain the amount of waterway which he must allow for his bridge, and this is often a difficult task.

When the banks are well defined and the river does not overflow; the question is comparatively easy ; but when, as is frequently the case in India the river channels hold mere rivulets or are perhaps dry in the hot weather, while in the rainy season, they are in high flood spreading over a wide expanse of country, the question becomes one of some intricacy as it is difficult under these conditions to distinguish the moving current from the backwater and to determine the velocities at which the water is flowing in different parts of the cross section. The Engineer seldom has an opportunity of seeing the highest floods himself and must therfore obtain his information from others. Even if he is so fortunate as to witness a flood himself he has probably not the means at hand at the time for measuring the sections and observing the velocities.

21. **Calculation of maximum discharge**—In drawing up a project for a bridge it is necessary to estimate the *maximum discharge* of the river or stream to be crossed in order to fix the dimensions of the bridge. In ordinary practice there are two methods of estimating the discharge: these should be used, when possible, as checks on one another.

(I) Measurements of the mean cross section of the highest flood and estimation of the mean velocity either (*a*) by application of Kutter's or Bazin's formula, or (*b*) by direct observation of velocity by means of floats or a current meter.

(II) Measurement of the drainage area, observations of the maximum rates of rainfall on this area and estimation of the amount which reaches the site of the bridge.

22. **Calculation of discharge from mean flood section and mean velocity**—By the first method the discharge is obtained by multiplying the area of the mean flood section by the mean velocity, either calculated or observed.

In order to determine the *mean flood section*, the following procedure should be adopted. First, determine by enquiry the height of the highest flood ever known and correct the information, if possible, by flood marks. Then take an accurate section of the river's bed at right angles to the course of the channel at the site of the proposed bridge and calculate the area contained between the highest flood line and the bed. Do the same at sections one mile above and one mile below the proposed site, and find the mean area by adding together the three areas thus obtained and dividing by three.

To find the *mean velocity* by calculation it is necessary to know the mean hydraulic mean depth and the slope of the bed or water surface. To ascertain these, proceed as follows. Measure the length of the undulating line of the river's bed in each cross section, viz., the wetted perimeter, and divide each area by this length; the quotients will be the *hydraulic mean depth* of each section. Add together the three mean depths so found and divide by three. The quotient will be the mean of the three *hydraulic mean depth* to be used in the calculation. Then ascertain by means of a levelling instrument the difference of level of the water surface at the margin at the upper and lower sections, i.e., the fall* of the water surface in the two mile length. This will give the slope in the form of a fraction of which the numerator is the fall and the denominator the length between the upper and lower sections, viz. 2 miles, both expressed in feet. Having thus ascertained the mean flood section and the slope of the water surface, the velocity is obtained from the formula $V=C\sqrt{rs}$ where r represents the Hydraulic mean depth in feet, s the slope as above explained, V the mean velocity in feet per second and C a co-efficient which, far from being constant, varies within a wide range and depends on the hydraulic mean depth and surface slope as well as on the rugosity of the channel. Bazin's formula for the calculation of this co-efficient is the simpler of the two in common use, but Kutter's is supposed to be more accurate and specially suitable for large rivers and irregular sections. The latter only will be given in this Manual; the former can be readily obtained, if desired, by reference to any text-book on Hydraulics. The expression for the value of the co-efficient C in Kutter's formula is unfortunately very complex. It is (adapted to English feet) as follows:

$$V=C\sqrt{rs}$$

$$\text{Where } C=\frac{41{\cdot}66+\frac{{\cdot}00281}{s}+\frac{1{\cdot}811}{f}}{1+\left(41{\cdot}66+\frac{{\cdot}00281}{s}\right)\frac{f}{\sqrt{r}}}$$

V is the mean velocity in feet per second, f is the modulus of rugosity of the bed and banks (its value varies for different materials), s is the slope of the water surface or bed, and r the hydraulic mean depth. The following table gives values of f for canals and rivers:

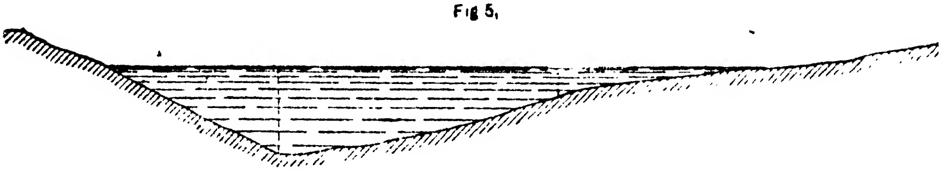
Description of bed and bank	f
Canals in very firm fine gravel	·020
Rivers and Canals in earth in perfect order and regimen, and free from stones and weeds	·025
Rivers and canals in earth in fair order and regimen, with occasional stones and weeds	·030
Rivers and canals in earth in bad order and regimen, with much stones and weeds	·035

* The fall is often so slight that it is necessary that this levelling should be done with the greatest care along both banks, if possible. The principal levelling should be done between bench marks on the banks at the upper and lower sections, and finally the water surface in the river should be connected across with these bench marks as nearly as possible at the same time and if possible by two different observers at the same instant.

The value of the co-efficient C will be found to depend largely on the value assigned to f; the value of f can only be assigned according to the judgment of the Engineer. To save the labour of calculation, the values of C for ordinary cases are given in a Table in Appendix A, See Example 4, paragraph 33.

23. In calculating discharges from velocities obtained by the foregoing method of calculation it should be borne in mind that, if the section is very irregularly as in Fig. 5, it is necessary to divide the whole section into two or more fairly regular figures and calculate the hydraulic mean depth, velocity and discharge of each figure separately to give an approximately correct result. The total discharge will be the sum of the discharges of the different figures.

Fig 5.



24. If the velocity is to be ascertained by actual observation, the following methods should be adopted. If the stream is a small one, it will be sufficient to mark out two lines across it 50 feet apart and to take several observations of the time occupied by a surface float, placed in the axis of the stream, in traversing the distance between the lines. The mean of the observations gives the maximum surface velocity and the mean velocity can be taken approximately to be four-fifths of the surface velocity, if the latter is less than 3 feet per second or nine-tenth if it is greater. Observations should be made at three cross sections at least, one at each end of the reach and one midway at the site of the bridge. If the stream is a large one, velocity rods should be used Two wires furnished with pendants at similar intervals, are stretched across the ends of a 100 feet run. A hollow rod, sufficiently long to extend from the surface nearly (but not quite) to the bottom, is started above any pendant at the upper section, and the time taken to traverse the distance from that pendant to the corresponding pendant at the lower section is noted with a stop watch. Experiments show that such a rod measures very approximately the mean velocity of the vertical section in which it moves. The rods are of different lengths to suit variations of depth, the proper length to use at any pendant being previously determined by soundings at the cross sections. The rods are cylindrical hollow tubes, 1 inch in diameter, of sheet tin loaded at the base with

iron and adjusted with shot so as to float with about 2 inches out of the water. The top is closed and marked with a tuft of cotton wool. For further information on this subject see the report by Major Allan Cunningham, R. E. on Hydraulic Experiments at Roorkee, 1881.

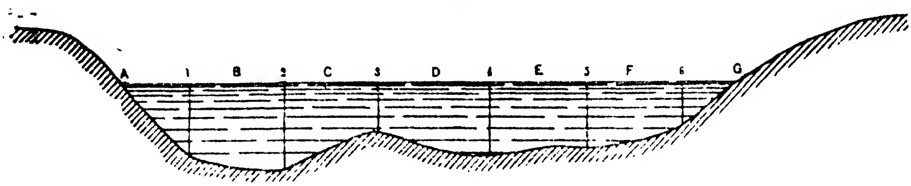
25. Another method of finding the velocity by direct observation is by the use of velocity meters, of which there are many varieties in the market, but none of these is really of much service in the ordinary operations of gauging river floods. The best-known type is the Current Meter. This consists of a small screw, like the propeller of a steamship, which is operated by the current, and records the number of its revolutions on a counter. The screw is kept with its head against the current by a large vane placed in the rear. The instrument is lowered on a rod to the position required and can be put in or out of gear at will. Owing to friction, lip of the screw, etc., the number of revolutions of the screw is by no means proportional to the velocity. The relations between the two called the "Equation of the Meter" is ascertained by *direct experiment* on each instrument. This is usually done by dragging the instrument repeatedly *at different velocities* through a known distance in still water and noting the time by a stop watch. From the average number of revolutions per second recorded by the meter on each vertical of a cross section, the mean velocity on that vertical is calculated by applying the "Equation of the meter." The objection to the instrument is that the relation between the number of revolutions recorded and the actual velocity is liable to be affected by the moving parts becoming clogged with silt.

26. With regard to observation of flood velocities by means of floats or meters, it should be noted that, for flood discharges of river liable to rapid rise and fall, the opportunity very rarely occurs of carrying out a sufficient number of velocity measurements in time of high flood to form a good estimate of the discharge, so that, for such rivers the formulae already given for the calculation of velocities from the hydraulic mean depth and bed slope are almost the only available means of flood discharge measurement.

27. In calculating the discharge from velocities obtained by observation, the follewihg method is adopted : Suppose the river cross-section to be divided into convenient areas AB, CD, DE, EF, etc., of lengths l_1 l_2 etc. (Fig 6). Verticals, 1, 2, 3, etc., mark the centres of the areas, and on these verticals the velocities are teken. The mean depth at each vertical is ascertained by sounding, and the velocity

v is determined by observation. The discharge in any area is (ld) v and the whole discharge is K(ldv), the mean velocity being $\frac{K(ldv)}{A}$ where $A=K\ (ld)$.

Fig. 6.



28. **Calculation of flood discharge from Catchment Basins**—The other method of estimating the discharge is by calculating the run-off from the catchment basin, above the site of the bridge and its embanked approaches, during the heaviest known rainfall. The main data required for this calculation are the area of the catchment and the maximum rate of rainfall. The area can readily be found in most cases from a contoured survey map, as its boundary is defined by a watershed, the drainage of which on the inner side flows to the basin in question while that on the outer side discharges into other adjoining basins. Part of the rainfall fails to reach the point of final discharge at the site of the bridge owing to absorption and evaporation. The amount lost depends chiefly on the nature of the soil, the fall of the country, and the shape of the basin. This subject will be found fully discused in the Manual of Water Supply, in the chapter on Gravitation Schemes, which is applicable here, *mutatis mutandis*, it being understood that the run-off required for bridge calculations is that due to the *maximum* rate of rainfall on a previously saturated soil, and not, as in the case of storage reservoirs, that deduced from the rainfall of the whole year.

The maximum rate of rainfall for a given basin must be obtained from the register kept at the nearest recording station the period taken for estimating the rate ranging from two hours for very small catchments to 24 hours for large ones. The rate of discharge from the basin will not, however, be directly proportional to the fall because—

(i) Very heavy rainfall is often only local, and sometimes occurs over a limited area not greater perhaps than 5 square miles round the station. Equally heavy rainfall may occur at other points in the basin, but possibly not at the same time.

(ii) The larger the basin the greater the probability that the flow from the ground near the point of discharge will have ceased before the flow from remoter portions has had time to come in.

To provide for the proportionate reduction thus required, for large areas, various empirical formulae have been proposed those chiefly employed being—

Ryves' formula $Q = C\,M^{\frac{2}{3}}$.

Dicken's formula $Q = C_1\,M^{\frac{3}{4}}$

Where Q is the discharge in cubic feet per second, M the area of the catchment in square miles, and C, C_1 local co-efficients depending on the rainfall, soil and slope of the district. Values of these co-efficients suited to particular districts are best deduced from measured maximum flood discharges from known catchment basins. For catchments in the plains of Upper India, C is generally assumed to be 675 C_1 825.

Mr. J. Craig, M. I. C. E., has framed a formula* for the discharge from a catchment area which appears to give reliable results—

$$S = 184{\cdot}2\ B \times \log \frac{8L^2}{B},$$

Where S = the sectional area in square feet of maximum flood at the discharging point.

B = Mean width of area in miles,

L = Mean length " "

The maximum flood discharge can be obtained in the usual way by applying the total sectional area found by the formula to the cross section of the stream at the discharging point and ascertaining therefrom the hydraulic mean depth, and by measuring the declivity of the bed.

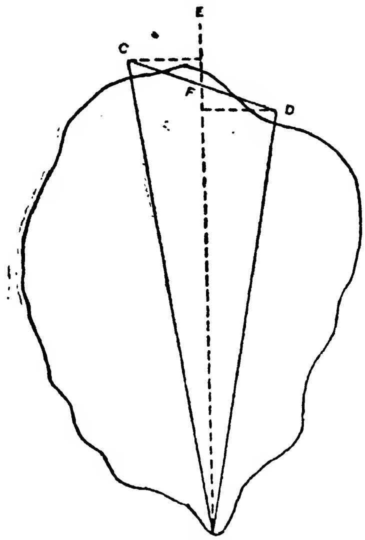
This formula is founded mainly on a consideration of the shape of the catchment area, and the accuracy of the results has been demonstrated by numerous results worked out by the author.

To obtain B and L, the perimeter, or ridge line, of the catchment area should be rectified by means of straight lines as CD which, with the

*See " Professional Papers on Indian Engineering ", articles LXXX, Vol. III and XCV, CVI, Vol. IV, Third Series.

distances CA and DA, will divides the area into triangles with their apices at the point of discharge.

Fig 7.



For each triangle L=the mean length, and B= half the sum of the perpendicular distances from C and D to AF or L, *i.e.* its mean width so that B × L gives the area of the triangle ADC.

The sum of discharges from all the triangles into which the figure is thus rectified will give the total discharge of the basin.

29. As a general rule it is a good practice, when calculating discharges by catchment basin formulae, to test the results by comparing them with the actual discharges of other basins under similar conditions and with those obtained from flood section and bed slope formulae. To do this it is necessary, first, to have the usual data of sections transvers and longitudinal, of the river's bed. Then to determine by a few trials the flood level which would give the discharge required by the catchment formula. This should be compared with the reputed flood level. If they differ materially, the latter should be again investigated. After this it will be safe to adopt whichever result gives the highest flood level.

30. No formula, however ingeniously constructed, can cover the entire range of conditions presented by large catchment basins. A snow-fed river, with a long course in the hills and then passing through an arid tract, can hardly be compared with one rising from springs in the plains and flowing throuagh a highly saturated country ; the rainfall is rarely uniform over a large area, and the length of a basin has a controlling and regulating influence on the floods passing off it. Perhaps the most difficult cases to determine are those in which the floods that,

previous to the throwing up of embankments, passed with an almost imperceptible flow over a wide expense of nearly level country are checked and have to be concentrated at the lowest depression; here sufficient waterway must be provided, and records are not available to check the results of formulæ. In such cases the demarcation of the various basins must be complete and accurate, and careful observations should be made to determine the highest rate of flow off the area per square mile when saturated by long continued rainfall.

31. Waterway of bridge openings—Having ascertained the discharge to be provided for by the methods above described, the next point for decision is the clear waterway to be allowed in the bridge openings to pass this discharge safely without dangerous scour. The safest width of opening would *prima facie* appear to be that which gives the full unobstructed waterway of the river, but this would not only be very expensive and quite unnecessary but, in the case of large wandering rivers, it would be positively prejudicial to the proper control of the river by means of Bell's Bunds, or other forms of modern training works, which aim at leading the water uniformly, or nearly so, through all the opening of a bridge across the whole width. We are therefore obliged to run some risk from scour by confining the floods to narrower bounds. This, in firm soils, causes heading up or *afflux*, and the velocity through the bridge openings is affected in certain proportions by the height of this afflux. The beds of rivers in soft soils are continually undergoing movement and change, the amount and nature of which depend, first on the soil of which they are composed, and second on the velocity of the water. The question to be settled then is to what depth does this river movement extend with certain velocities of current. Experience alone can be our guide in deciding this question. It is now generally accepted in Indian practice that a mean velocity exceeding 5 to 6 feet per second is dangerous to bridges whose foundations do not rest on firm soil or are not carried to great depths. This may appear to be a small velocity to cause much damage, but observation has afforded ample comfirmation of the correctness of this limit of velocity in rivers with sandy beds. Foundations in such river beds are not safe for higher velocities unless carried down either to a hard stratum or to a depth well beyond the limit of scour which is now generally assumed to be about 50 feet in sand in ordinary cases. It may be much more under special conditions.

32. The following formula gives the amount of afflux caused by obstructions in the river's course.

$$h = \left(\frac{V^2}{58 \cdot 6} + 0 \cdot 05 \right) \left\{ \left(\frac{A}{a} \right)^2 - 1 \right\}$$

Where V = Mean velocity of river in feet per second previous to obstruction.

A = sectional area of river unobstructed in square feet.

a = Sectional area of river at obstruction in square feet under original water level, i.e., *neglecting the afflux.*

h = rise of water in feet, or *afflux*, caused by the obstruction.

Example—The width of the Thames is 926 feet. The sum of the widths of the waterways of old London Bridge was 236 feet. The mean depth of the river is 9 feet The normal velocity is $3\frac{1}{2}$ feet per second—

$$A = 926 \times 9$$

$$a = 236 \times 9$$

$$\frac{A}{a} = \frac{926 \times 9}{236 \times 9} \text{ 4 nearly.}$$

$$\text{Afflux} = \left(\frac{3 \cdot 5^2}{58 \cdot 6} + 0 \cdot 05 \right) \left(4^2 - 1 \right) = 3 \cdot 75 \text{ feet}$$ agreeing nearly with the actual result.

33. If the section of the river and its velocities can be accurately measured, the amount of waterway required for the bridge, so as not to cause too great an afflux whereby the normal velocity would be dangerously increased, can be easily ascertained. The velocity permissible in any particular case will depend on the nature of soil of the river bed and the depth of the foundation. In this connection it should be noted that the afflux calculated as above will only occur if the bed remains unchanged. In sandy soils the scour under the bridge will enlarge the waterway and reduce the afflux, and a higher velocity will be permissible when the design of the foundation is adapted to it.

Having the mean velocity of the natural waterway, that of the artificial waterway may be obtained from the following expression—

$$v = \frac{A}{a} . V$$

in which A, V are the area and mean velocity of the natural waterway, and a, v the same data for the artificial waterway *including* in a the increment due to the afflux caused by the obstruction).

(*N.B.*—The a here used differs from the a of the preceding paragraph in that it includes the increment due to afflux which is not included in the a of paragraph 32.)

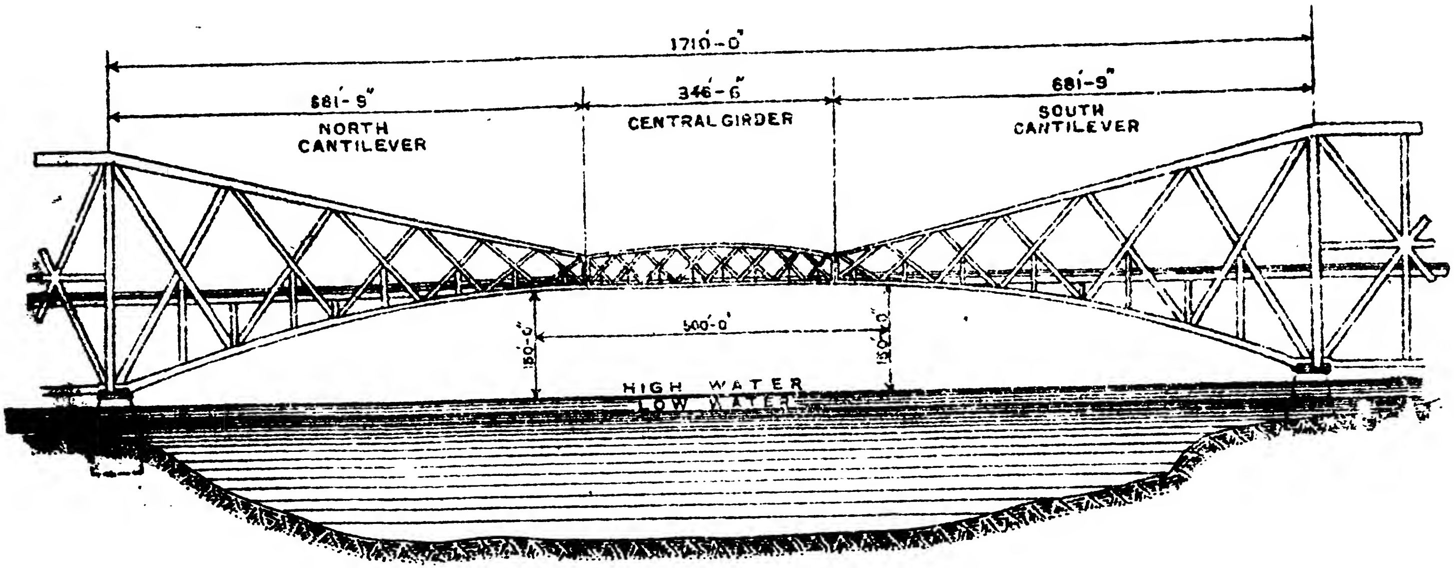
Example 1.—Given the area and mean velocity of the natural waterway A = ft. 4,600 square feet, V = 3·7 feet per second, to find the actual waterway a corresponding to a mean velocity of 5 feet per second—

Here $\therefore \frac{AV}{v} = \frac{4{\cdot}600 \times 3{\cdot}7}{5} = 3{,}404$ sq. ft.

This is up to the level of the afflux and includes it.

Example 2.—Suppose it had been proposed to force the stream through 8 openings, each 58 feet wide, the depth being 9 feet; then—

A SPAN OF THE FORTH BRIDGE



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Area of waterway, obstructed,

neglecting the afflux = 3 × 50 × 9 = 1350 sq. ft. = a,

Area of natural waterway = 4,000 sq. ft. = A.

$$\text{Afflux} = \left(\frac{8{\cdot}7^2}{58{\cdot}6} + 0{\cdot}05\right)\left(\left(\frac{4600}{1350}\right)^2 - 1\right)$$

$$= (0{\cdot}23 + 0{\cdot}05)\,(11{\cdot}61 - 1) = 0{\cdot}58 \text{ feet.}$$

Example 3—Suppose the average width of a stream is 100 feet, average flood depth 5 feet, with mean velocity of 3 feet per second; and that it is to be crossed by a bridge of 3 arches of 20 feet span each, then—

A = Area of natural waterway = 100 × 5 = 500 sq. ft.

V = Natural velocity = 3 ft. per second.

a = Area of obstructed waterway, neglecting afflux = 3 × 20′ × 5 = 300 sq. ft.

$$\therefore \text{h the afflux} = \left(\frac{3^2}{53{\cdot}6} + 0{\cdot}05\right)\left(\left(\frac{500}{300}\right)^2 - 1\right)$$

$$= (0{\cdot}15 + 0{\cdot}05)\,(2{\cdot}7 - 1) = 0{\cdot}55 \text{ feet.}$$

Depth of water including afflux = 5·55 feet.

Artificial waterway, including afflux or a = 3 × 20 × 5·55 = 333 sq. ft.

Hence 333 × artificial velocity = 500 × 3.

$$\text{V, or Artificial velocity} = \frac{500 \times 3}{333} = 4{\cdot}6 \text{ ft. per second.}$$

Example 4—The hydraulic mean depth calculated from three flood sections taken near the site of a bridge is 8·1 feet. The fall of the water surface is 1 in 1000. The river bed is in fair order, and not obstructed to any appreciable extent by stones or weeds. Find the mean velocity of the flood.

Data — r = 8·1 feet — See para. 22.

$$s = \frac{1}{1000} = {\cdot}001.$$

From the tables in appendix A, f being ·030, C = 71.

$\therefore V_1$ (mean velocity) $= 71\sqrt{8{\cdot}1 \times {\cdot}001} = 6{\cdot}39$ per second.

34. The springings of arches should in every case be *at least* 1 foot above the level of the afflux shown as resulting from the obstruction, and the lower booms of girders should be at least 4 feet above it. They may of course be as much higher as the height of the banks will allow or navigation requirements demand.

Design

35. Under ordinary circumstances, the design of a bridge, great or small, will be influenced by the nature of the material procurable, the skill of the workmen, the amount of money to be extended, and the nature of the river passed.

36. **Span**—After the foregoing calculations in regard to waterway have been made, the Engineer is in a position to determine the number of spans or bays of which his bridge is to consist. If the stream is gentle, its bottom not likely to be disturbed, and the foundations not likely to be deep and expensive, and if the materials are not good and the workmen are unskilful, the Engineer will probably choose a number

of small arches. If the river be subject to great floods, the bottom liable to erosion, and the foundations likely to involve great expense, he must procure good workmen and have few bays of large span, the superstructure being of steel girders or arches of brick or stone.

It is not, however, always possible to choose the span of a bridge. Circumstances may determine the span which alone is practicable, as when foundations can only be met with at particular spots, where certain breadths of span are required for navigation or other purposes, etc. Where a choice of span can be made, it is evident that the cheapest efficient one is the most desirable, and this may be assumed generally to be the longest efficient span which can be made for a sum equal to the cost of one of the piers on which it rests.

37. The cost of the piers per foot of waterway of the bridge is as the length of the span inversely and the mass of the piers directly. The mass of the piers must, however, be increased somewhat for a larger span to sustain the weight of the span and the load on it, so that the span cannot be increased indefinitely to reduce the cost of the pier per foot of waterway.

Also the cost per foot of waterway for the span alone in steel bridges increases in a ratio in excess of the span owing rather to the material of which the bridge itself is made than to the useful load it is to accommodate—a remark that applies with greater force to the larger spans.

Hence there is no advantage in increasing the size of the span beyond what is required to reduce the cost per foot of waterway for an efficient pier to a minimum, so that the cost of both pier and span per foot of waterway may *together* also be a minimum. The framing of a design in which the balance of efficiency shall be kept between the pier and the span must, therefore, be tentative in the first instance, for it is necessary to assume some breadth for the pier which it is supposed may be efficient for a given set of circumstances as regards the force of the floods, nature of the foundation, etc., and from such a pier to deduce the longest efficient span which can be made at a like cost. It is not improbable that the breadth of pier assumed may require modification when this is done. It may be considered too large for the deduced span, in which case a reduction can be made, and from the reduced pier a new span may be deduced, and the process repeated till we have attained a design which is fully efficient and which will at the same time give the cheapest bridge.

38. Summing up a few of the leading points, the following tend to economy in a bridge by shortening the span:

(1) Good foundations, necessitating less expense for efficient piers.

(2) Small height of the piers, reducing the quantity of masonry or material in the piers.

(3) Cheapness of the material of which the piers are composed.

(4) Greater strength of the material in the piers, reducing the quantity required.

(5) Cheapness of labour in constructing the piers.

(6) Any modification of design which gives the same efficiency with a less quantity of material in the piers.

Any of these reduce the sum for which an efficient pier can be made.

The following tend to economy in the bridge by lengthening the span:

(1) Excellence and strength of material in the span, admitting of higher unit stresses per square inch and spreading a given amount of material over a wider span.

(2) Cheapness of the material in the span, enabling a larger quantity to be used for a given expenditure in providing a larger span.

(3) Cheapness of labour for erecting the span.

(4) Any modification of the design of the span by which an increased, yet efficient, span can be got out of a given quantity of material.

Any of these tend to an efficient span of greater length for a given sum of money.

39. As regards the efficiency of a design, it appears that for deep rivers the *piers* should consist of a single member, and not of an assemblage of parts. The usual form given to masonry *piers* has undoubtedly a superior strength over all others, combining the greatest solidity with the greatest rigidity and stability to resist the action of the forces in the river and the vibration caused by heavy rolling loads on the bridge. Besides this the form admits of the greatest area of base for offering resistance to the flood. It does away with the necessity for the external connections required in steel structures, which are a source of weakness and expense in maintenance. Obstacles, such as trees, are not liable to be arrested by cross currents and eddies between the members as in the case of detached piles.

40. In the matter of the superstructure, it is evident that the character of the design must depend to a great extent on the span to be used and the comparative cost of the materials of which the bridge might be constructed.

Broadly speaking, the superstructure of a permanent bridge might be of brickwork or stone masonry, ordinary concrete, reinforced concrete, cast-iron ribs, or wrought-iron and mild steel girders.

41. Wooden bridges are common in all parts of India where timber abounds and where cheapness and rapid construction are important. Wood has its objections in being less durable than brick, stone or iron, but, if covered in and protected from the weather, it may be made to last a considerable time. It must, however, be admitted that structures of timber, however skilfully designed and executed, cannot be regarded in any other light than that of temporary engineering expedients, doomed inevitably in the course of time to be superseded by those of a more stable and permanent character.

42. Brickwork, masonry, and concrete are usually employed for culverts and for bridges of moderate span. Concrete, being liable to crack from expansion and contraction due to extreme variations of temperature and requiring very close supervision in mixing and laying to be successful, is seldom used (unless reinforced) for spans exceeding 40 feet. Brickwork and masonry arches are employed chiefly for road and canal bridges and for canal aqueducts; also to some extent for railway culverts up to 20-feet span. Bridges having a superstructure of brick or stone are more durable than any other class of bridge, but they involve heavy foundations and, in India, their spans are limited to about 70 feet in ordinary practice, though in Europe arches up to 280 feet span have recently been constructed for railway bridges, examples of which will be found in Chapter V on arched bridges. Reinforced concrete is now also coming into use for bridges, and several large spans have recently been constructed of this material in the United Provinces in places where lightness of superstructure is an important object, see Chapter VI.

43. For very large spans the only suitable material for the superstructure is iron or mild steel. In the hills, where transport is difficult and expensive, the suspension type of bridge is the most suitable for roads. Cast-iron has been used occasionally in the form of arched ribs for bridges of large span. The Southwark bridge across the Thames in London, built in 1819, has a central arch of 240 feet span and 24 feet rise. But it is not usual in these days to employ this material to any extent for bridge-building. Its tensile strength is small as compared with

that of wrought-iron and mild steel, while the strength of the latter to withstand tension and compression is nearly equal for both stresses. Cast-iron does not therefore lend itself so well to economy and lightness of design, and is consequently not so suitable for girders or cantilevers of large span.

44. Further particulars in regard to the design of foundations and superstructure will be given in the following chapters dealing with the different types of bridges.

45. Sunk bridges without embanked approaches, designed for total immersion during high floods, are occasionally permissible in particular localities where the banks are very firm, the river bed is hard rock, and strict economy is desirable, but as a rule they require special protective works and can hardly be recommended for ordinary use.

46. The case of flood openings in railway and road embankments is one demanding particular attention in the preparation of designs for bridges over large rivers subject to considerable overflow across wide reaches of flat country on the banks. They should be avoided as far as possible, as pointed out in paragraph 10. The design of such openings are decided on considerations differing materially from those applicable to bridges over rivers with well defined channels. The determination of discharges from nearly flat, and previously unobstructed, basins has already been referred to. The width of flood openings to be allowed in any particular case will evidently be governed by the maximum height of afflux permissible without detriment to the agricultural and sanitary conditions of the country covered by overflow in high floods. In this connection see also Mr. Spring's recommendation as regards Bell's bands in paragraph 15.

CHAPTER III

TEMPORARY BRIDGES

47. Before entering on the subject of permanent bridges it will be useful to consider the temporary expedients which are so often resorted to, especially in India, to facilitate the execution of permanent works or for crossing rivers and streams where want of time, money or skill prevents the employment of more durable methods.

48. **Causeways or "Irish bridges"** are the simplest variety of such expedients on roads. Many Indian watercourses are dry, or nearly so, except during the rainy season. In some of the very dry districts of the Punjab, the watercourses often contain no water for eleven months in the year. The beds of such streams are generally passable without a bridge, but they require to have the banks cut down to an easy slope, say 1/20 for roads, and the bed of the stream paved for a sufficient width to reduce the labour of draft through heavy sand or sand and water. For sandy rivers in the plains, it is often sufficient, for 2nd class roads, to lay "fascines" or bundles about 9″ diameter of grass, etc. across the track over the sand, the ends being secured by a longitudinal fascine securely pegged down by stakes. For more important road-crossings, a paving of stone, brick or concrete is laid, strong enough to withstand the rush of water during freshes. The thickness of the paving depends on the nature of the bed and the force of the stream in flood. If the soil is very soft and yielding the paving material is laid in a trench 4 to 5 feet deep as in the example on page 29, but in ordinary sand the usual practice is to lay a paving of stone, set in lime mortar 6 to 9 inches thick, on a substantial bed of concrete. In the latter case, to guard against scour and prevent the paving being undermined, a dwarf wall is built on each side, the down-stream wall being made two to three feet deeper than the upstream one which is usually two feet deep.

Though classed with temporary bridges in this chapter, it should be noted that in cases where the funds available are not sufficient for the construction of regular bridges, causeways are sometimes constructed as permanent crossings on roads, and even on feeder railways, over shallow streams which are dry for the greater part of the year.

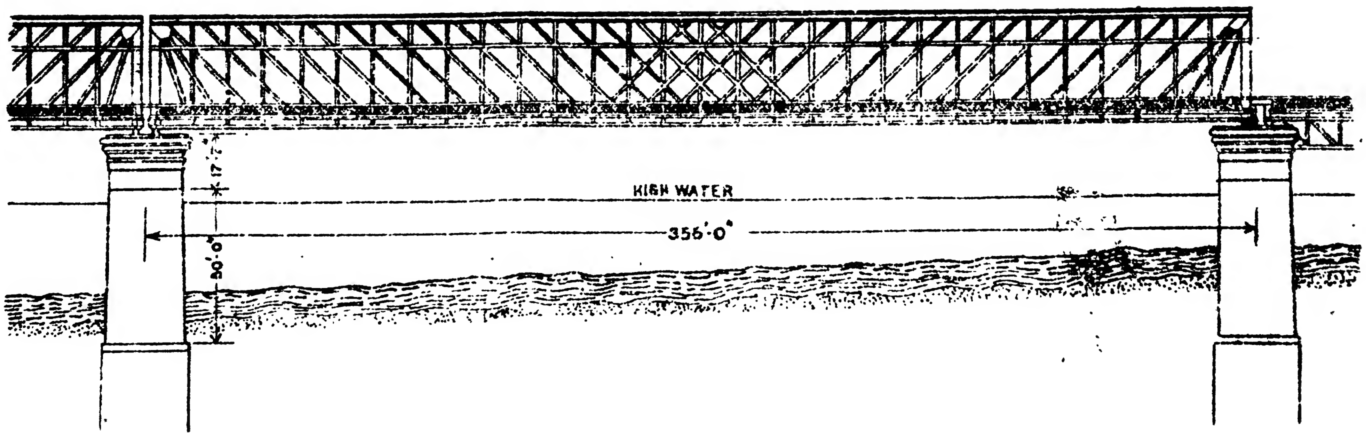
49. Before designing a paved causeway for a crossing, the river bed should be carefully examined to see if there is any evidence of its being much scoured by floods or of its being subject to extensive movement by

MULTIPLE (TRIPLE) INTERSECTION LATTICE GIRDER

WHIPPLE TYPE

(DUFFERIN BRIDGE, BENARES)

ELEVATION



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the current, for, under such conditions, it must be remembered that a causeway will act as a weir and probably alter the regimen of the river seriously.

50. The following is a description of a substaintal type of paved causeway across a large sandy river in the plains where the Grand Trunk Road crosses the river Soane.

The line of the causeway having been marked out, common jungle piles were driven in two parallel lines, 17 feet apart, to a depth of about 15 feet; the sand having been excavated with shovels in the ordinary way between these piles down to the water level or a little below, bamboo frames with palm tree leaf, mats were forced down behind to prevent the sand from the sides slipping into the excavation. The remainder of the sand below water level was excavated to the required depth by means of the ordinary well sinker's *jham*, worked from temporary stages on either side of the excavation.

Fig 8 PLAN.

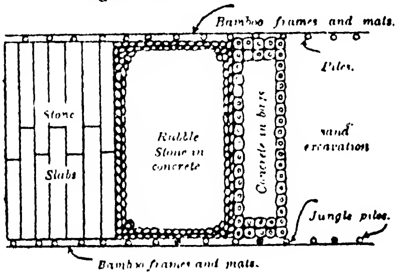
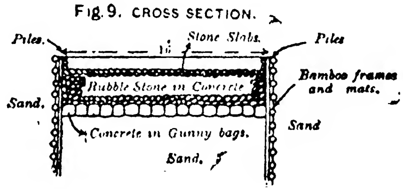


Fig. 9. CROSS SECTION.



A layer of gunny bags filled with concrete composed of two parts Soane shingle, one part *surki*, and one part *kankar* lime, was then set, as closely packed as possible, over the whole bottom of the excavation.

Over this a layer of rubble stone, 2 feet 6 inches, deep, set in concrete (of the same proportions as above) was placed, and on this the roadway was formed of roughly cut stone slabs 1 foot thick, from 1 foot to 1 foot 6 inches broad, and alternately 9 and 7 feet long to break joint with each other; all irregularities of the lower surface of the slabs being carefully packed up with rubble stone, the joints made as norrow as possible without actually dressing the stone, and thoroughly filled up, grouted and pointed with the best *kankar* lime and *surki* hydraulic mortar. Any considerable roughness or irregularity of the surface was then chipped down and levelled by stone-cutters, no attempt being made, however, at anything approaching fine dressing of the whole or any part of the surface of the stones, which would have been objectionable through causing the road to be dangerously slippery.

51. **Temporary bridges** are usually made of timber, but use is ometimes made of steel wire and hemp rope, iron rails, pipes, telegraph poles, etc. The fastengings in common use for such bridges are hemp or wire ropes, iron spikes of the plain or dog variety, bolts, screws, and nails. The temporary bridges described in this chapter will only be such as are suited for carrying carts, foot-passengers, and pack animals. They may

conveniently be classified broadly under the following heads and sub-heads:

(1) Bridges of which the road-bearing beams are supported at intervals on the bed of the stream by means of (*a*) trestles, (*b*) piles, (*c*) cribs or crates.

(2) Bridges spanning the whole gap without any intermediate supports by means of (*a*) timber trusses, (*b*) timber cantilevers (*c*) suspension cables.

(3) Floating bridges (1) flying, (2) with road-bearers supported at intervals upon (*a*) boats, (*b*) pontoons, (*c*) rafts of barrels, etc.

These will be described below in the above order.

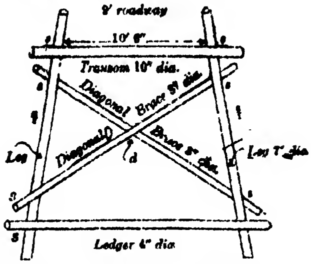
52. (1) (*a*) **Bridges on trestles**—Trestles as piers are the most generally useful, and are applicable to most cases where the bed is fairly firm; they are easy to make and erect, economical of material, and do not require very long or large pieces up to heights of 20′. Over that height they become heavy and unwieldy and require special arrangements for erecting them. They are unsuitable for rapid streams, on stony beds, running more than four miles an hour or more than 6′ deep. The span of each bay may be any length to suit the size of the timber available. Trestle bridges may be made of squared or round timber.

It is very necessary to prepare a careful cross-section of the bed at the site of the bridge in order to make the trestles of the correct height. If the bottom is very uneven, soundings should be taken at each side of the bridge at the places where each trestle will stand. The heights of the trestles should provide for a camber of 1/30 up from each bank to the centre of the whole span to allow for any possible settlement that may take place subsequently.

Trestles may be two-legged, three-legged or four-legged. *See* Figs. 10, 11, and 12. It should be noted that all the dimensions shown on timbers, etc. in these figures are for deodar timber, and for bridges with a 9′ roadway carrying a live load of 560 lb. per foot run, with trestles 15 feet high, and bays of 15′ span.

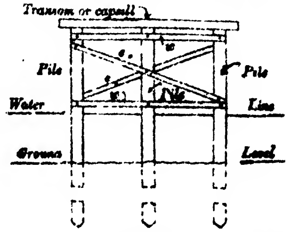
Two-legged trestles are the most generally useful and consist of two legs with an inward slop of about 6/1, a transom fixed horizontally across the legs near their tops, a ledger fixed across their feet to keep them from spreading and sinking (sometimes omitted), and two diagonal braces. The legs may be cut off above the transom, but are usually left 5′ longer to provide standards for the rails. The size of the legs should be calculated in each case for the load they will have to carry; a diameter of 8″

Fig. 10. TWO LEGGED TRESTLE, OF ROUND TIMBERS LASHED.



Lashings omitted. All junctions lashed with wire or rope; "square" lashings at s. s. and "diagonal" lashings at d.

Fig. 14. PIER OF PILES. FRONT VIEW FASTENINGS BOLTS OR SPIKES.



ww. walling pieces, double, both sides of piles. ee. braces.

Fig. 11. THREE-LEGGED OR TRIPOD TRESTLES, ROUND TIMBER LASHED.

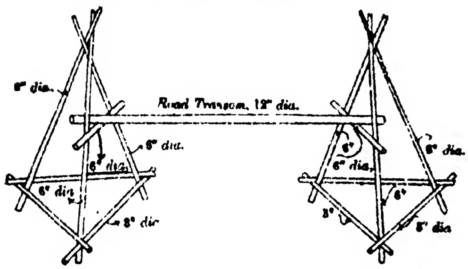


Fig. 15. CRIB PIER OF SLEEPERS SIDE VIEW.

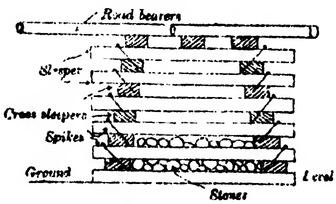


Fig. 12.

FOUR-LEGGED TRESTLE OF SQUARED TIMBER, SPIKED TOGETHER.

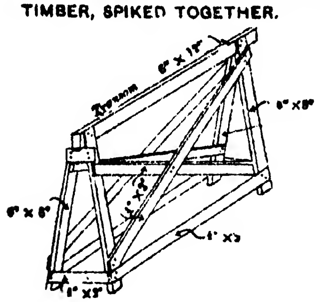
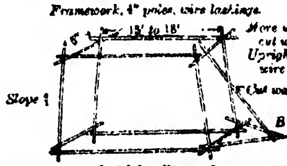


Fig. 16

CRATE PIER OF BRUSHWOOD, FILLED UP TO TOP WITH STONES WHEN IN SITE.

Framework, 4" poles, wire lashings.

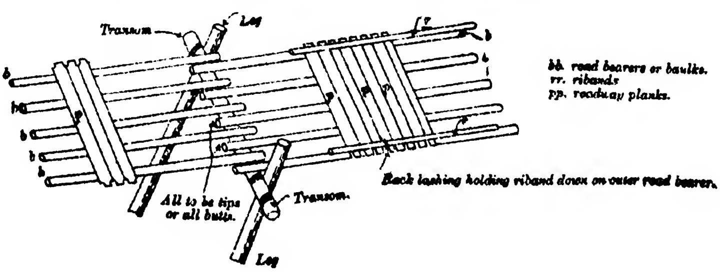


More uprights add... sides and cut water covered with hurdling. Uprights to be tied diagonally with wire at every 3' height

Bottom covered with hurdling to form tray for stones

Fig. 13.

ARRANGEMENT OF ROADWAY ON A BAY OF A BRIDGE.



bb. road bearers or baulks.
rr. ribands.
pp. roadway planks.

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will, as a rule, be found to be enough for a 15′ bay to carry a 9′ roadway with a live load of 560 lb. per running foot at a height of 20 feet.

If the trestle is made of squared timber, as in Fig. 12, the pieces are spiked together; but if rounded timbers are used, the fastenings are best made of wire or wire rope as hemp rope lashing stretch under stress after a time and have to be tightened up frequently. If the timber available be small, the legs and transoms may be made of two or three spars lashed or spiked together.

Trestles are erected at site on the dry bed or in very shallow water, but where the depth of water is considerable they are made up on the bank, taken out on a raft, and tipped up into position (by guys) with their feet weighted, or, if heavy, they are placed in position by a swinging derrick on the raft.

Three-legged or tripod trestles are sometimes used where the bed consists of mud, but they are not suitable for very uneven bottoms and are difficult to place in position. They can be made of light material, and stand without external bracing. Two of these trestles are required for supporting one transom, *see* Fig. 11.

Four-legged trestles are useful at intervals in a long bridge of two-legged trestles to give longitudinal stability, but they are not generally used, as they are heavy structures and difficult to place in position.

3. **Superstructure**—The superstructure of all bridges of Class (1) whether supported on trestles, piles or crates, is practically the same down to the transoms of the piers supporting the road-beams. For an ordinary road-bridge this is best arranged as in Fig. 13. The planks are laid across the axis of the bridge, supported on road-bearers or beams laid across the span parallel to the axis. This planks are secured by two ribands laid on top of them, one over each outer road-bearer to which the riband is lashed, or bolted down. The ribands also act as wheelguides or wheelguards. If squared road-bearers are used, the planks can be spiked down to them. The ends of the road-bearers rest on the pier transom. Planks for ordinary traffic may be 2″ thick and 9″ to 12″ wide. If supported by road-bearers at 2′ intervals, these will carry any cart or gun with an axle-load not exceeding $1\frac{1}{2}$ tons. For bridges under much wheeled traffic, the planks should be thicker (up to 4″) to stand the heavy wear. When planks thicker than 2″ are not available, longitudinal planks are sometimes fixed over the others under each wheel track to take the extra wear. If planks cannot be procured, round "ballis" may be used touching one another. Earth

should not be laid on top of planks or "ballis" as it rots the wood and adds greatly to the dead load on the bearers. Grass may be spread on top of the planks to deaden the noise for animals.

The minimum clear width between wheelguards or ribands should be 9′ for wheeled traffic, and, if vehicles are to pass each other, 16′; for pack animals the minimum clear width should be 6′, and, for foot traffic, 3′.

Every bridge should have a "camber", or rise in the middle of 1/60 of the total span to improve its appearance and to allow for any settlement that may take place after erection.

The shore ends of the road-bearers of all bridges should rest on a stout broad timber laid in firm ground across and under them to prevent any one bearer sinking into the earth independently of the others and to keep them all at one level.

Hand rails should be 3½′ to 4′ high, made of scantlings about 4″ diameter.

The bays may be of 10′, 15′ or 20′ span according to the size of timber available. If spans greater than 20′ are used, the road-beams will probably need trussing.

54. (1) (*b*) **Bridges on pile piers**—Piles from very strong and stable piers, and are suitable for deep streams with muddy bottoms, but, for large piers, require long straight timbers, take sometime to erect, and require a pile-engine to drive them. *See* the chapter on Foundations for a description of pile-engines. For foot-bridges over shallow muddy streams they can be very quickly made, the piles being driven by a man with a "maul" or wooden mallet; the larger kind are often used for important road bridges of a *quasi*-permanent type. An xample of the latter is given in Plate I.

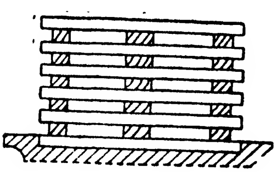
The pier may consist of two to five piles which are driven vertically in a line across the pier from 3′ to 5′ apart, and when sufficiently driven are cut off level at the proper height. The transom or capsill is then spiked on to their tops. They are usually braced together by diagonals, and by horizontals called "waling pieces," *see* Fig. 14. Piles may be of square or round timbers 6″ to 10″ diameter or more. The feet are shaped to a point and, if of soft wood, are shod with iron. The heads are usually bound with an iron hoop. The extent to which piles should be driven to stand any given load without undue settlement is explained in the chapter on Foundations. The superstructure of pile bridges is practically the same as that of trestle bridges which has been described in full detail in paragraph 53.

55. (1) (*c*) **Bridges supported on crib or crate piers—Cribs** or crates are easily made of cheap material and require no skilled labour. They make very stable piers and are often useful for rough jobs when time is the main consideration, but they obstruct the water-way a good deal and require a lot of material. They are particularly suitable for swift shallow streams with round boulder bottoms. They are sometimes used at intervals in a long trestle bridge as longitudinal stiffeners.

Cribs are generally made of sleepers fastened with trestles or spikes, or of rounded logs notched together, *see* Fig. 15. The bottom of the crib is planked to form a tray. The crib is then floated out to the site, and the tray filled with stones till the crib rests on the bottom. The length of a crib should be at least $1\frac{1}{2}'$ plus the width of the roadway.

Crates are easily made with poles or branches. Good examples of these may be seen every year at Hardwar over the branches of the Ganges at the time of the April fair. They are best made of four strong uprights at the corners with horizontals at the top and bottom (as in Fig. 16) fastened with wire lashings. Other uprights are added between the corners ones, and the sides are finally filled by hurdles or brushwood. The bottom is covered to form a tray, and an inclined pole is added at the upstream end to form a cutwater. As the weight of the superstructure is taken by the stone filling, the crate should have the uprights tied across at different levels diagonally by wire to prevent its being bulged out by the lateral pressure of the stones. The crate is made as above on dry ground, floated out to the position it is to occupy, and then sunk by being filled up with stones. The length of crate should not be less than $1\frac{1}{2}'$ plus the width of the roadway.

Under this head should be mentioned the temporary accommodation bridges made on railways to pass trains at slow speed over a gap at a road or drainage crossing while the permanent piers and abutments are being built. These are usually made of open stacks of sleepers built up from the ground to the level of the joists or girders carrying the rails. The sleeper stacks are placed in the spans between the piers to allow the latter to beb uilt up conveniently at the leisure.



56. **Trussed Bridges**—These may be made of simple trussed beams as in Figs. 17, 18, 19, or of strutted beams as in Figs. 20, 21, or of girders proper as in Figs. 22,23.

Trussed beams and trussed girders may be made of wood alone or of wood and iron bars. The latter is a good combination as wood can be used for the compression members and iron for tension.

The trussed and strutted beams shown in Figs. 17 to 21 are suitable for spans up to 30′ and 45′ as marked on each figure. Trussed girders may be made up to 60′ span without much trouble, but over that they become unwieldy and difficult to launch; Fig. 22 is an open braced girder which is suitable when iron rods are available, and Fig. 23 a plate girder type which can be easily made of planks and nails when iron is not available.

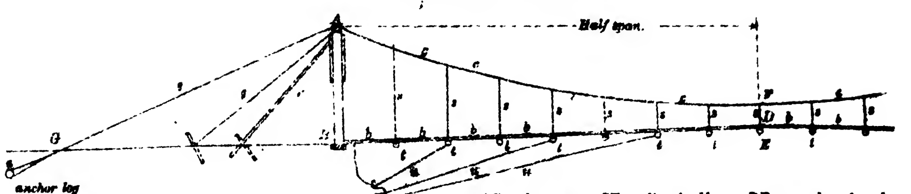
If the available headway will admit of it, the roadway should be carried on top of the girders as this arrangement is convenient for fixing the cross weather bracing between them. The depth of a girder should be as great as convenient from maximum of $\frac{1}{6}$ span to a minimum of $\frac{1}{15}$; it is usually $\frac{1}{8}$ to span $\frac{1}{10}$. Large girders are best made on the bank and launched on rollers, a suspension cable carried over shear legs on the bank being used to support the forward overhanging end as the girder rolls across the span. The overhanging end is sometimes supported by a tackle from shear legs erected on the far bank.

The longitudinal timbers of trussed beams may be composed of several lengths butting over the heads of the struts and fished together by bolts, and they may be made of several thicknesses of full depth, side by side, bolted together and breaking joint. Several trussed beams may be used side by side at short intervals to cross the span, as the roadway rests on the top of them. Trussed beams with two struts should always be counterbraced in the middle for use in a bridge as they are subject to moving loads.

Strutted beams are simple and strong and have been largely used with pile piers in *quasi*-permanent road bridges on the Murree-Kashmir Cart-Road in the Punjab. Very little iron is necessary and that only for fastenings. The road beams should rest on blocks at the angles of the frames or, better still, on a transom connecting all the frames. *See* Fig. 21. The depth should not be less than $\frac{1}{4}$ of the span in Fig. 20 or $\frac{1}{6}$ span in Fig. 21. The struts should be braced by ties BF, CE unless the span is very small.

The calculations of stresses and sizes of the different members of the supporting frames described above are of a simple nature and are not given in this chapter as they will be readily made by students who have gone through the College Course of Applied Machanics.

Fig. 27
"SLING" OR "COMMON" SUSPENSION BRIDGE
6 FEET WIDE ROADWAY



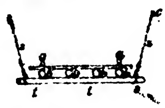
AC. = ½ span. CF = dip of cable. DE. = camber of roadway.

AB. frame or pier. g. back guy and r. strut of frame
GAF. main suspension cable c.c. BD. roadway.
AG. backstay (continuous with main cable). s.s. sling supporting. t.t. road transoms
b b. road bearers. u.u. stiffening guys underneath to bank.

Fig. 28. PLAN OF Fig. 27. SHOWING WIND GUYING ARRANGEMENTS.

bb. road bearers, continuous over 2 bays; 4 per bay is better.
aa. ribands breaking joint with road bearers, and continuous over 2 bays.
ww. side windguys to bank
sc. side (horizontal) catenary cable, and sg. its guys to transoms.

Fig. 29. CROSS SECTION AT A PAIR OF SLINGS.



aa. ribands. 8" dia. or 8" × 4".
NB.—4 road bearers are shown in Fig. 29 and only 3 in the plan Fig. 28
bb. 4½" dia. or 8½" × 4"
tt. 6½" dia. or 6" × 6"

WOOD FRAME FOR PIERS OF NEW GILGIT. SUSPENSION BRIDGE, SPAN 368' FOR PACK ANIMALS.

Fig. 30. FRONT ELEVATION.

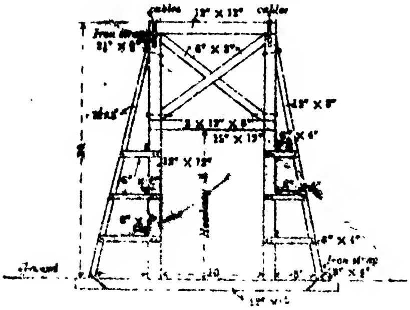


Fig. 31. SIDE VIEW

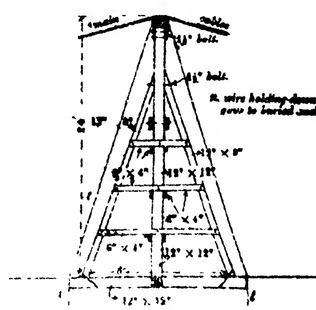


Fig. 32. LOG ANCHORAGE SECTION.

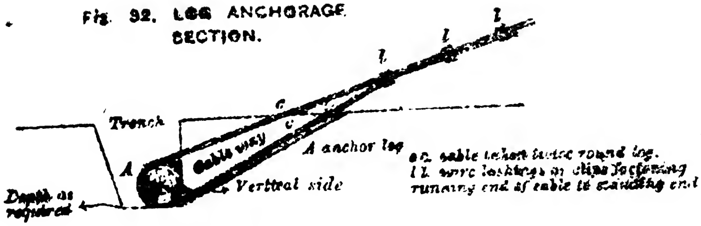
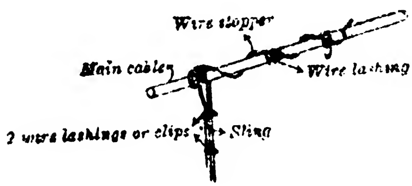


Fig. 33



The "stopper" prevents the sling slipping down cable

Fig. 34. SECTION.

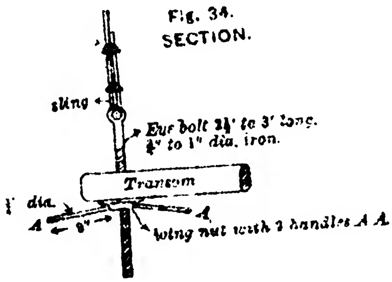


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Fig. 35. ARRANGEMENT OF A FLYING-BRIDGE WITH SUSPENSION CABLE.

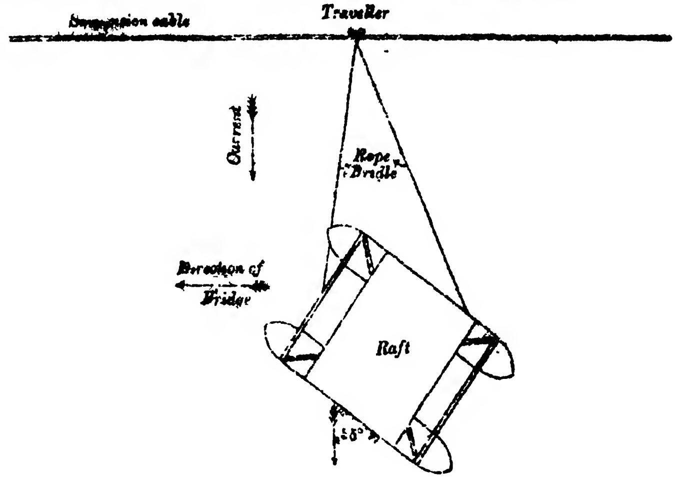


Fig. 36 ARRANGEMENT OF FLYING-BRIDGE WITH SWINGING CABLE

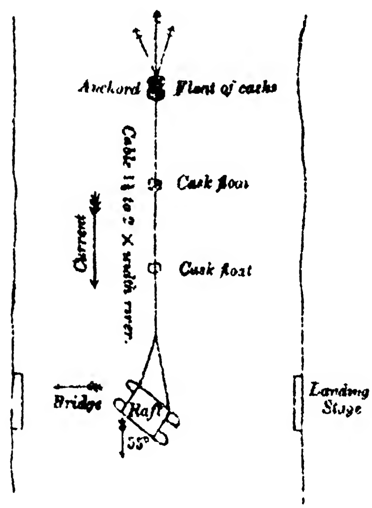
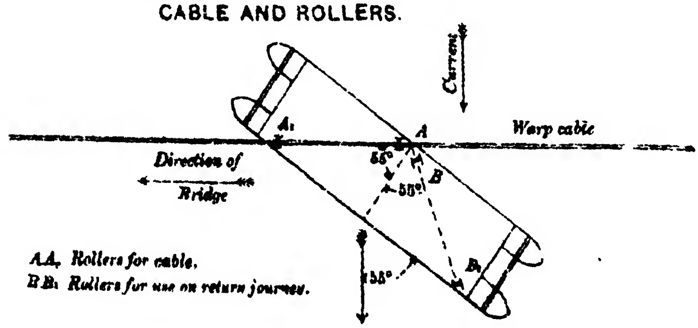


Fig. 37. ARRANGEMENT OF FLYING-BRIDGE WITH A WARP CABLE AND ROLLERS.



P.S.U.P (R) 4, S.P.S. 1847-1960.

sort of cradle in which the passenger sits is slung on the rope and pulled across by means of ropes worked from either shore. In other cases the bridge consists of three ropes, one for the feet and two others for the hands of the traveller, kept asunder by pieces of wood placed at intervals. On the other hand, iron or steel suspension bridges of very large spans and of a permanent kind have been constructed in Europe and America over important roads in places where the cost of other forms of bridges would have been prohibitive. As examples of these may be mentioned the well-known Clifton suspension bridge of which a full description will be found in the Proceedings of the Institution of Civil Engineers, Vol. XXVI, and the Brooklyn suspension bridge which is described in Vol. X of the *Engineering News* pp. 20, 241, and 252. The former stretches across the gorge, bordered by steep cliffs, through which the River Avon flows below Bristol. It has a span of 702 feet between the supports of its chains, and the roadway is at a height of 248 feet above water level. The bridge is 30 feet in width and is carried by three chains on each side having a dip of 70 feet. The Brooklyn bridge at New York has a central span of 1,600 feet between the supports of the cable, and two side spans of 930 feet, affording a headway in the centre of 135 feet above high water. The roadway, 85 feet in width, comprises a central elevated footway, two wire rope railways, and two carriageways on the outer sides. Suspension bridges have been generally regarded as unsuitable for railways owing to the excessive oscilation produced by passing trains.

64. **Flying bridges**—When sufficient funds are not forthcoming or sufficient material is not available to make a regular bridge across a stream, a boat or a raft may be used as a ferry. Ferry boats are usually rowed or poled across. The action of the current on the boat or raft may also be utilised to move a ferry boat across a stream. A ferry so worked is termed a "flying bridge."

There are three methods of working flying bridges: (1) by suspension cable, (2) by using anchors and "swinging cables" and (3) by using a "warp." These are shown in Figs. 35, 36, 37, which explain themselves.

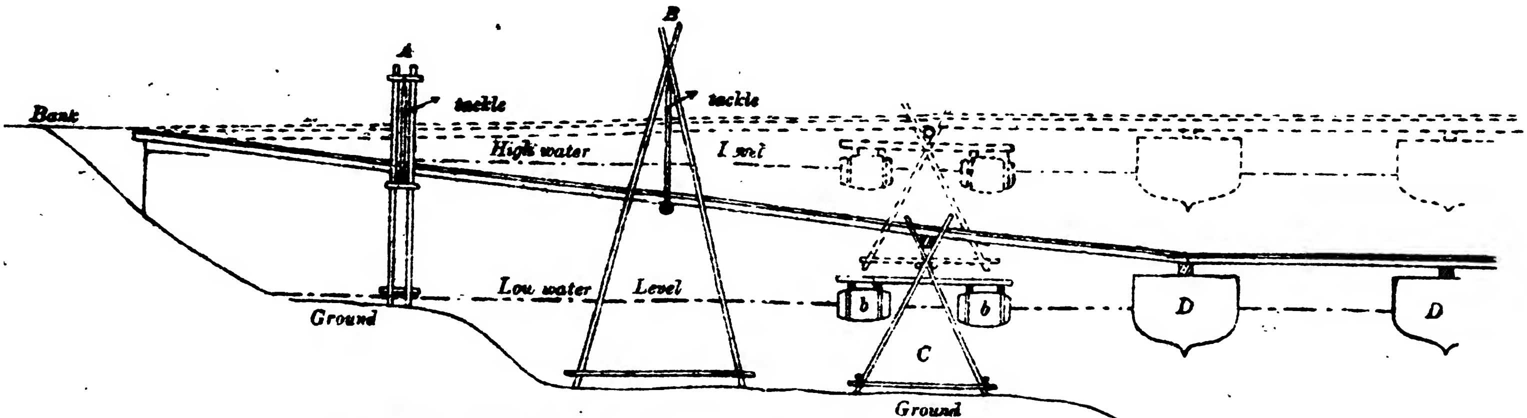
The first method is much the best. A post or shear-leg, with raking struts to secure it in a vertical position, is set up on each bank at a height sufficient to keep the centre of the cable well above flood level, and the cable is suspended from the tops of these posts. These piers should be pitched as far from the banks as possible to keep the steepest parts of the dip of the cable away from the working length of the cable on which the traveller will run. The cable should be stretched as taut as its strength will permit. The traveller should have two wheels as shown in Figs. 38, 39. Two lines are attached to the traveller and to the raft as shown in Fig. 35, the length of the longer line being varied as required to keep the length of the raft inclined approximately at an angle of 55° to the direction of the stream. For returning, the lines are moved to the other side of the raft.

In a swing bridge the length of the cable should be from one and a half time to twice the width of the river, and if a long one is used it should be kept out of the water by being supported on intermediate floats. The end of the cable is secured to a short mast fixed in the boat or raft about one-third of its length from the bow, or the cable may be secured to a rope bridle as shown in Fig. 36. If a bridle is used, it should be about three times the length of the boat to allow of its being let out to give the boat an inclination of about 55° to the current. The cable requires a firm anchorage, two or three anchors of 300 lb. being generally necessary. In a very rapid current, the cable may be anchored to both banks instead of to a float in the middle of the stream. In this case four landing-places and two cables will be required, one of the cables being taken across in the boat for the return journey. When the boat reaches the far bank, it touches at a lower landing-place downstream and is hauled up to the upper one from which it starts back on its return journey.

In the third method a cable or "warp" is stretched across the river as tightly as possible and the boat or raft runs along it, the cable running through rollers on the raft at A and A, the rollers AA and BB being used alternately to keep the raft at the proper angle to the current in going and returning.

65. **Boat bridges**—Boats form good piers, especially in swift rivers but do not bear grounding and generally require a good deal of carpenter's work to fit them up. Each boat should have enough buoyancy to support the heaviest load that can come on it. The buoyancy is the weight of water displaced by the boat up to a foot or fifteen inches of the gunwale, less its own weight. Only $\frac{9}{10}$ of the calculated buoyancy should be taken as the available buoyancy, a greater margin being allowed in rough water. In calculating the load to be carried by buoyancy, no allowance is necessary for live loads as distinguished from dead weight to be carried by the bridge, i.e., live loads need not be multiplied by $1\frac{1}{2}$ to convert them to dead loads. A central saddle beam should be provided in each boat along its longitudinal axis to support the baulks carrying the roadway. The upper surface of the saddle should be slightly

SHORE END OF A FLOATING BRIDGE FOR VARYING WATER LEVEL



A. Two-legged Trestle with double legs and adjustable transom.
B. 2 tripod trestle. (3rd leg not shown)
C. Four-legged trestle resting on ground at low water, and carried by a 2 pier raft of barrels.
b b. at high water.
DD. Floating piers of boats. etc.,
The position of roadway at high water is shown dotted

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above the two gunwales and of length about two feet more than the width of roadway. In large boats with strong sides, the saddle may be supported on crossbeams laid directly on the gunwales, but in most boats it is necessary to support the saddle upon the keelson by a longitudinal trestle. With large boats of 20 tons capacity and over, and when only short baulks available, the method illustrated in Plate III may be adopted.

66. Floating bridges, whether made of boats, pontoons or rafts, are secured in position either by anchors or by tying each pier to a strong cable or chain stretched across the stream ; or if the river is narrow, by two cables from each pier to bollards on the banks. The second method with a suspension cable or chain is much the best in rapid currents, and in very swift stream with strong bottoms no other is possible. Anchors do well enough for firm or muddy bottoms. As a rule, every second pier has an upstream anchor, but in rapid streams every pier should have one. A few downstream anchors are also necessary to provide against strong wind blowing up the river. To ensure a secure holding, the length of anchor cable should not be less than 10 times the depth of the stream with a minimum of 90'. The weight of anchors required is from 10 to 20 lb. per foot run of bridge. The pull due to a floating bridge pier may be taken to be roughly,* in pounds the product of the immersed cross mid-section in square feet multiplied by the square of the velocity of stream in feet per second multiplied by a co-efficient varying with the shape of the immersed portion. Several values of the co-efficient (Beaufoy's) will be found in Molesworth's pocket-book, 26th edition, page 559.

67. If the water at the shore ends is too shallow to float piers, the shore bays may be supported on trestles, etc., or a causeway of crib-work or earth built out up to the floating portion. In a river subject to sudden and rapid rises and falls, the shore ends need special arrangements. Generally, trestles with transoms adjustable in height, or trestles to carry the roadway at low water with floats under the bay to rise and carry it at high water or a combination of these, will meet most cases. *See* figure on opposite page. If the end bays are arranged to slope slightly down towards the bank at high water, the gradient down from the bank to the bridge at low water will be much improved.

68. To allow boat traffic to pass the bridge, a passage or "cut" is often required. This "cut" is made by removing one or more bays of the bridge which are formed into an independent raft and allowed to float downstream and on to one side of the passage thus made. *See* Fig.

* Trautwine's Engineer's Pocket-Book, 1883 edition, page 571.

40. The movable raft has a set of special short baulks on each side to connect up its roadway with that of the two standing portions of the bridge. *See* Fig. 41.

The boats nearest to the shore on each side being more particularly affected by weights moving on and off the bridge on account of the sloping approaches, two of the strongest and most capacious vessels should be selected for the ends of the bridge.

The baulks or road-bearing beams are laid on the boat saddles or trestles in separate spaces, cleared to receive them, in which the beams work as the boats move in the water. They should not be deeper than ⅔ breadth as lateral strength is necessary to resist the horizontal stresses to which they are exposed. The flooring boards should be 2½ or 3 inches thick, laid across the beams and covered with several inches of stable litter or straw.

Where proper anchors are not available, wooden cribs filled with brick or stone, or a rough wooden anchor surrounded with strong netting which can be filled with boulders, *kankar*, brick, or large blocks of stone may be used.

69. The following is a detailed description of a boat bridge superstructure designed by the late Major-General Sir Alexander Taylor, K.C.B., R.E., for the Ravi, Indus, and Chenab rivers on the Lahore-Peshawar Road which was found to be very efficient and economical. *See* Plate III.

The bridges are maintained throughout the year. They have a catenary curve with versed sine equal to ⅛th the span, and are held across the stream by a 1¼ inch crane chain on the upstream side and a 1-inch chain on the down stream, the ends being secured on both banks in concrete blocks by heavy baulks of timber. Every alternate boat is anchored on the downstream side to preserve the bridge from being blown upstream by a storm, or displaced by floods.

They consist of the following component parts :

1. **Gunwale pieces**—*See gg*—(Figs. nos. 1, 2, and 5). Each is a single piece of timber 14½ feet long and 8×8 inches scantling, secured to the gunwale by two bolts of round iron of ¾ inch diameter.

It has notches of one inch in depth, to receive both the trussed and the cross-beams and two deep notches, to receive the stiffening beams, of such dimensions as to bring the top surface of the trussed and of the stiffening beams to a level. Fig. 5 shows all details.

2. **Trussed beams**.—*See kk*—(Figs. 1,2). They may be each of a single beam 80 feet long and 7×7 inches scantling, or each may be built of four pieces bolted together by round bolts of one inch diameter, as shown in the drawings. Each beam runs in over the

TRAVELLER FOR FLYING-BRIDGE CABLE

Fig. 38 SIDE VIEW.

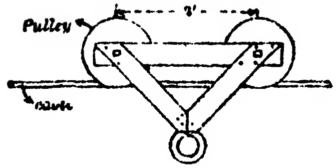


Fig. 39.

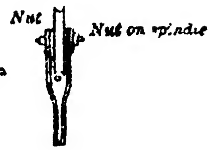
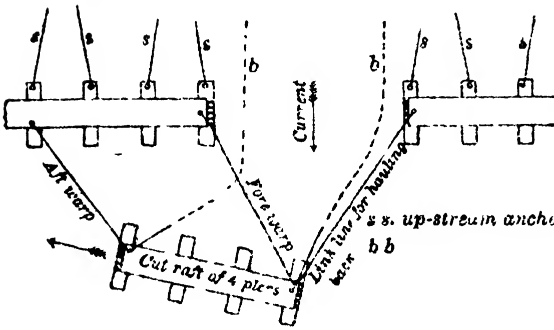
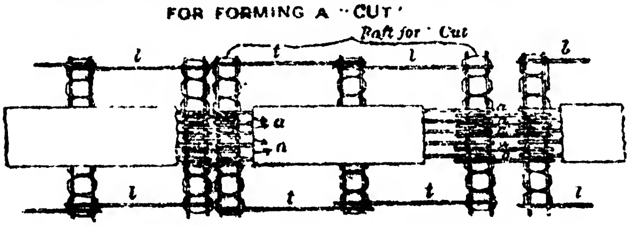


Fig. 40. FORMING A "CUT" OR OPENING IN A FLOATING BRIDGE.



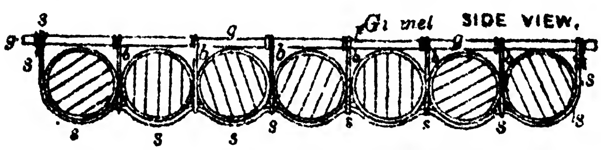
s s. up-stream anchor cables of bridge piers.
b b „ cut raft.

Fig. 41
BARREL PIER BRIDGE WITH A RAFT OF 3 PIERS FOR FORMING A "CUT"



t t tie baulks connecting ends of piers.
a, a short "cut" (roadway) baulks or road-bearers

Fig. 42
LASHING CASKS TO 2 GUNNELS TO FORM A BARREL PIER.



s s. *Slings, 2½" rope.*
b b *Braces, 1½" rope.*
g g *Gunnels about 4"×5" or 5" dia.*

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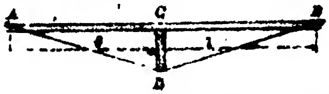
TRUSSED AND STRUTTED BEAMS.

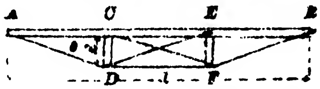
FIG. 17 TRUSSED BEAM WITH 1 STRUT



CAD = θ $\qquad \cot\theta = \frac{l}{2d} \quad \operatorname{cosec}\theta = \sqrt{\frac{l^2+4d^2}{2d}}$

Depth d may be from $\frac{1}{8}l$ to $\frac{1}{12}l$. spans up to 30′.

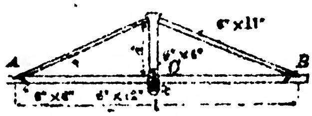
FIG. 18. TRUSSED BEAM WITH 2 STRUTS



CAD = θ $\qquad \cot\theta = \frac{l}{3d} \quad \operatorname{cosec}\theta = \sqrt{\frac{l^2+9d^2}{3d}}$

AC should = $\frac{1}{3}l$ Depth d may be from $\frac{1}{12}$ to $\frac{1}{8}l$
Span l may be up to 4′

FIG 19. "A" OR "KING-POST" TRUSS OF WOOD ONLY.

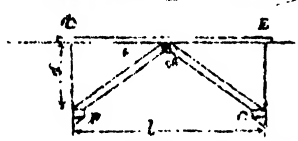


If wood only is available use "A" truss as it is used up to 30′ span, but is not so good as Fig. 17. as the long pieces AD, ED are struts in compression in Fig. 19. To avoid putting transverse stress on the tie beam in addition to the tension; suspend a transom c under the centre C from one truss to the other of the span; and use separate road-bearers, between the 2 trusses, supported at centre of span on this transom C.

The stresses (direct stresses) in this form are the same as Fig. 17 but reversed
Sizes of timber shown are for 30 span, to carry 550 lbs. live load per foot run deodar timber

FIG. 20. STRUTTED BEAM FOR SPANS UP TO 30 FEET

DE = l. DAB = θ DB = d,



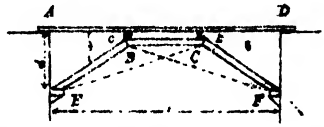
$\cot\theta = \frac{l}{2d}$

$\operatorname{cosec}\theta = \frac{\sqrt{l^2+4d^2}}{2d}$

d should not be less than $\frac{1}{8}l$

c is a block against which the struts AB, CA bear or it may be a transom c joining the heads of all the struts in the span.

FIG. 21. STRUTTED BEAM WITH STRAINING BEAM FOR SPANS UP TO 45 FEET



AB should = BC = $\frac{1}{3}l$.
should not be less than $\frac{1}{2}$ AB = $\frac{1}{6}l$

$\cot\theta = \frac{l}{3d}$

$\operatorname{cosec}\theta = \sqrt{\frac{l^2+9d^2}{3d}}$

The struts BE, FC bear against the "straining beam BC"
To prevent cross breaking stress on BC; support the road-bearers upon two transom c c. resting at the joints B, C of all the straining beams in the spans, which will then be, under compression only.

FIG. 22 WOODEN GIRDER FOR 50' SPAN, 9' ROADWAY, TO CARRY LIVE LOAD 560 lbs PER FOOT RUN. DEPTH, ⅛ LENGTH.

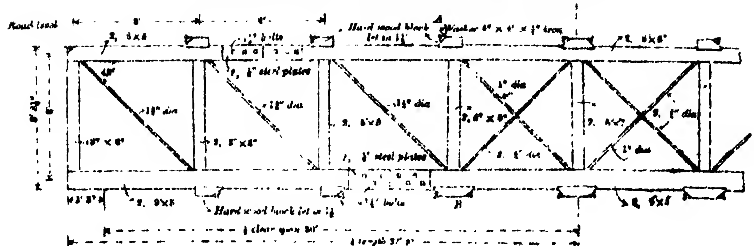


Fig. 23 WOODEN PLATE GIRDER FOR 50' SPAN, 9' ROADWAY, TO CARRY LIVE LOAD 560 lbs. PER FOOT RUN. DEPTH, [illegible] LENGTH.

The web planks (5" × 1") on inside of girder (not shown) are [illegible] angles to those on 1½" outside. On both sides they are placed [illegible] 12 inches apart at [illegible]. Each [illegible] 4, 5 inches wire nails at each end.

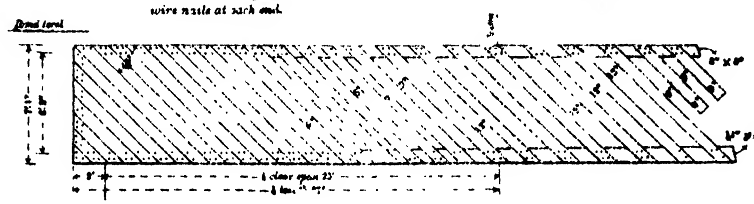


Fig 24. CANTILEVER BRIDGE STIFFENED BY WIRE TIES

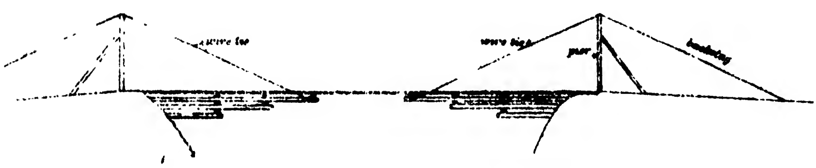


Fig 25. "RAMP" SUSPENSION BRIDGE. 8' ROADWAY

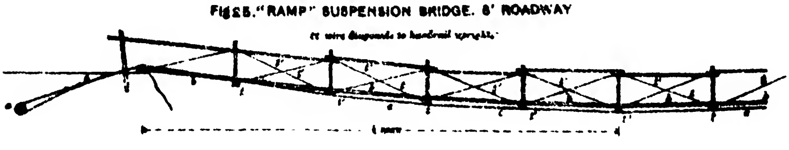
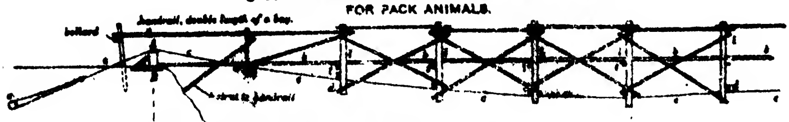


Fig 26. "TRESTLE" SUSPENSION BRIDGE. 8' ROADWAY FOR PACK ANIMALS.



ll. legs of trestles. tt. transoms. dd. ledgers resting on main cables cc.
AB. low pier. a anchor log. tt. transom 6" × 6". bb road-bearers, 5½" × 6" for 10' bays, 4 per bay

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57. **Cantilever Bridges**.—These are often used in hilly countries where long beams or logs are easily procurable. They have been made up to spans of 120′. No skilled labour or iron-work is required, wooden pegs being used to keep the timbers in position, nor is heavy tackle necessary for erection. Cantilever bridges of long span are hardly stiff enough for heavy traffic, but can be improved by adding wire stays. *See* Fig. 24.

These bridges consist of layers of logs on each bank, each layer projecting 5′ to 10′ beyond the ends of the layer below till only a gap of 15′ to 20′ remains, over which a row of road-bearers is laid to complete the bridge. The projections should not exceed 10 feet, and, to utilise the strength of tapering spars to the fullest, the butts may be laid outwards as the maximum bending moment in a cantilever always occurs at the point of support. The cantilevers should not in practice be laid horizontally as shown in Fig. 24, but at a slope up towards the centre to give a greater depth of loading on the buried ends, and cross-transoms should be placed between each row of cantilevers over the points of support. The inner ends of all the layers of logs are held down by anchoring them to other logs buried in the dry stone masonry of the abutments, or by weighing them with a cribwork of timber and stones. The former construction is shown in Plate 2, which gives details of a bridge of this kind built by the late Colonel A. M. Lang, R.E., over the river Sutlej at Wangtu. The abutments of this bridge were of dry stone masonry, substantially bonded throughout by frames of deodar wood running through the structure and not only strengthening the masonry but keeping the ends of the cantilevers rigidly fixed.

Cantilever bridges of a permanent type will be found described in Chapter VII on iron and steel bridges.

58. **Suspension Bridges**.—These have the advantage of only requiring cables and material in small light pieces. They can therefore be made of very large span in places where, owing to difficulties of transport, no other form of bridge would be possible. Temporary suspension bridges have been made in Chitral and Gilgit of spans of 320′ and more to carry pack animals and foot passengers.

A suspension bridge consists essentially of two cables or sets of cables, one on each side, hanging in a curve or catenary from which the roadway is supported. The cables pass over supports on the abutments and are kept taut by being secured to anchorages fixed in the banks beyond the abutments. The cables may be of any strong flexible material. Wire rope is the best as it does not stretch much; iron chains are good but not so reliable as steel wire rope; hemp ropes may be used but they do not last long and stretch so much as to be very troublesome. Ropes made of ratan cane, twigs or bamboo strips, have all been used on occasions when nothing better was procurable. The supports on which the cables are carried are usually timber frames (like trestles) for large bridges, and simple posts or shear legs for small spans; if the bridge is to be *quasi*-permanent, masonry piers may be used. The anchorages may be logs buried deeply in the ground or covered by a large stack of stone, or iron jumpers let into rock and in a sloping position more or less normal to the direction of the rope. The roadway is usually of wood and the length of each bay between the vertical supports not more than 6′ to 10′, as the bridge being flexible it is desirable to load the cables at numerous points close together.

The great disadvantage of a suspension bridge is that it lacks stiffness under a moving load and, with an advancing load covering only a part of the span, the roadway sinks at the loaded end and is liable to vertical distortion. This can be avoided to some extent by forming the handrails into an open braced girder which helps to distribute partial loads over the whole cable and stiffens the structure, by supporting the transoms of the roadway near each end by struts from the bank, or by tying back these transoms to the tops of the piers. Modern practice favours the method of stiffening by means of braced handrail girders which are hinged in the middle in some cases for still better distribution of partial loads (*see* the example referred to in paragraph 62). High winds also have a very disturbing effect on such bridges, causing them to sway sideways, or to rise vertically in deep gorges where strong currents up to the valley are apt to catch the underside of the roadway. These effects are generally counteracted by wire, or wire rope, wind ties securing the end transoms to the banks on each side.

59. Suspension bridges are of various types and designs to suit different localities, (1) the ramp bridge, *see* Fig. 25, in which the roadway is carried on the cables themselves, (2) the trestle type, *see* Fig. 26, in which the roadway is supported above the cables on trestles which stiffen the structure to some extent but add considerably to the quantity of timber used, and (3) the sling bridge, *see* Figs. 27, 28, in which the roadway is suspended by wire, chain, or rope slings from the cables. The first two types are uncommon and will not be further referred to. The sling bridge is the most useful and will be described in detail.

60. The main cables of a sling bridge are in two sets, one over each side of the roadway. Each set may be one or more cables, side by

side. The dip of the cables at the centre of the span (below the top of the piers) is $\frac{1}{10}$ to $\frac{1}{15}$ span. The less the dip the stiffer the bridge, but the greater the the stress in the cables; $\frac{1}{10}$ to $\frac{1}{12}$ span is usual for temporary bridges. The slings or suspenders support the transoms of the roadway and are spaced 5′ to 10′ apart, *see* Figs. 27, 28, 29. A camber of $\frac{1}{60}$ span should be allowed in the roadway. The handrails should be fixed on wooden uprights strutted outside to continuations of roadway planks projecting beyond the others. The piers should be 3′ to 5′ longer across the bridge than the width between the centre pair of slings to give a horizontal splay in the cables and stiffen them against swaying. The anchorage should be fixed at such a distance from the pier that the cable will make equal angles on each side of it, i.e. the angle BAG will be equal to the angle BAC in Fig. 27. If the site will not permit of this, the piers will have to be strutted, or guyed behind, to prevent overturning. Each pier usually consists of two vertical legs joined by a "capsill" on top and a "groundsill" under the feet, the legs being braced at the top by a horizontal and two diagonal braces. The legs should be side-strutted to take any side stress which the cables may bring on the trestles. *See* Figs. 30, 31. Where the cables rest, the capsill is rounded and covered with sheet-iron to prevent the cables cutting into the wood and to allow them to slide freely over the pier. For temporary bridges over 100′ span, the roadway should be narrow to keep down the weight——say a width of 6′ or so.

61. As young Assistant Engineers are often required to put up suspension bridges in places where skilled labour is not often available, the following note explaining the ordinary method of erection will be useful in this Manual:

To assist in the erection of the bridge, and for convenient conveyance of stores from one bank to the other, a temporary shear-leg is erected on each bank and a light service cable, with a traveller pulley on it, is stretched over them across the gap. The pier trestles are then raised either by foreguys from the far bank or by means of props and strut, and when in position they are secured by backguyes and backstruts. A single spar is then raised vertically and lashed temporarily so the capsill near one of cable seats. A pulley block tackle* is secured to the top of the derrick spar a few feet above the capsill. A rope bridle having been fixed to the cable at about the place where it will rest on the pier, the lower block of the tackle attached to the derrick is hooked into the bridle and the cable raised and dropped into its place on the capsill. All the main cables are thus placed in position, and their ends made fast to the anchorage by passing the end twice round the anchor log or beam and fastening the free end to the cable itself by three clips or by wire lashings. *See* Fig. 32.

* A tackle consists of double or treble pulley block, one at each end, and running rope between them.

Earth is then filed in over the anchor and rammed. The far ends of the cables are now passed round the other anchor and temporarily "seized" in place, the earth being filled in except at the cable groves. The cables are then raised to the top of the second pier in the same way. The next thing to be done is to haul in the slack of the cables to the required dip. For a small bridge this can be done with a simple rope, but a large bridge requires the use of running tackle. A strong rope sling is made fast to the standing part of the cable near the second anchor with a tackle on it and a second tackle to the running end. Both tackles are hauled on and the cable is raised a little higher at the centre of the span than the final dip required. For a final dip of $\frac{1}{10}$ the cable should be raised to about $\frac{1}{12}$ but the allowance to be made depends actually on the span, the dip, and the flexibility of the cable. It is necessary to raise the cable higher at first in order to allow for the stretching of the cables and the slight yielding of the anchorage when fully loaded. The ends of the cable are then finally seized to the standing part as in the first anchorage.

The slings and the transoms are now suspended from the cables as shown in Figs. 33, 34, the former having previously been cut to length and marked with paint at the proper places for final fixing.

For the convenient fixing of slings and transoms a "traveller" is usually made by putting a spar about 5" diameter across the top of the main cables. A rope is attached to each end of it and passed over the top of the pier so that the spar can slide on the cables and be held in any position. From each end of the spar a rope ladder is suspended, supporting a horizontal broad plank at a convenient height below the cables so that workmen can stand on the plank to fasten the transoms or climb up the ladder to fix the tops of the slings to the cables. It is convenient to have a traveller at each end of the bridge.

When all the slings and transoms are fixed, two road-bearers are put across each bay with four or five planks on them, and the slings are then finally adjusted to get all the transoms at the right height. When this has been done, the roadway is completed as usual.

62. The maximum tension on the cables occurs at the points of support over the piers and is equal to $\frac{Wa}{2}\sqrt{1+\left(\frac{a}{4d}\right)^2}$ where w=total load carried by the cable, a=span in feet and d=dip in feet. The strains on the other members are simple direct or transverse strains and are easily calculated. *See* Molesworth's or Trautwine's pocket-book of engineering formulae for convenient methods of calculation. A good example of a sling suspension bridge with fully detailed calculations will be seen in Punjab P. W. D. Paper No. 59 by Mr. H. C. Granville, Superintending Engineer, on the Dihar Bridge in the Punjab.

63. The above descriptions apply to suspension bridges ordinarily built by Engineers on important roads in the hills, but rougher and cheaper expedients have sometimes to be adopted to cross mountain torrents on unimportant lines of communication, or for preliminary road operations. Among these may be mentioned the *rope bridges* often seen in the Himalayas. These consist in some cases of a single rope stretched across the stream and made fast to trees on either bank, a

boat one foot, and rests on the gunwale piece in a notch one inch deep.

3. **Stiffening beams**—*See aa*—(Figs. 1, 2, 4 and 5.) These are stout beams of 10×6 inches scantling placed, as shown in the plate, two to each boat. Each should, if possible, be a single piece of timber 26 feet 8 inches long, but may be in two pieces scarfed together. They are let into the gunwale pieces to such a depth as to bring their upper edges and those of the trussed beams to a true and even surface.

4. **Cross beams**—*See bb*—(Figs. 2, 4 and 5) are 7×7 inches scantling and 13 feet 2 inches long and rest on the gunwale pieces in notches 1 inch deep. They merely rest on the gunwale pieces and are not fastened to them.

5. **Stiffening planks**—*See cc*—(Figs. 2, 4). Each plank consists of two pieces. One main piece 14 feet long and 12×3 inches scantling, and a minor piece (spiked to it as shown in Fig. 4) 12 feet long, 6 inches wide, and of such depth as to compensate for the difference in thickness between the trussed beams and the stiffening beams (*kk* and *aa*) which in the Plate, is 3 inches. They are placed, as shown in the Plate near the ends of the stiffening beams, and in every case immediately under a roadway plank. Each stiffening plank is tightly lashed at each end to the roadway plank immediately above it, so as to allow no play whatever, by means of a chain.

6. **A stiffening chain** which is shown in detail in Fig. 3 (*ee*) and as regards its general application, in Figs. 1, 2 and 3. Such a chain is passed round each projecting end of every stiffening plank and of the roadway plank immediately over it; the hook with which it is provided at one end is passed into the nearest link, and everything is then brought home by the use of a rough wedge *hh*. The centre of each chain is secured to the under part of the stiffening plank by a stout staple, which passes entirely through the plank, and is clenched on the upper side. The object is to prevent the chain from being lost in the confusion which often attends the dismantling of a bridge in a hurry, and perhaps in the dark and in heavy weather.

7. **Rough wedges**—*See hh*—(Figs. 1, 3 and 4) may be about 18 inches long. Their use is obvious.

8. **Ordinary roadway planks**—*See dd*—(Figs. 1, 4, 5) are 14 feet long, 3 inches thick, and as nearly 12 inches wide as may be practicable. The width for six inches at each end is to be reduced by three inches, as shown in Fig. 9. Each plank is to be furnished with two hard wood cleats 3″×2″ securely spiked on—*See* same figure.

9. **Railing planks**—*See ff*—(Fig. 15). Each plank is from 17 to 17½ feet long, and they may be placed at any convenient distance apart. The railing bars may be bullies of about five inches diameter, supported on trenails of 1½ inch diameter of hard dry wood.

70 **Pontoon bridges**—These are superior to boat-bridges and more expensive, but very similar in their construction and general arrangements. The following is a description of the pontoon bridge at Agra, which was in use for many years and formed the only means of crossing the Jumna at the place before the completion of the iron girder road and railway bridge, *see* Plate IV :—

The Agra bridge is supported on cylindrical sheet iron pontoons P.P. The total length of a pontoon is 30 feet 8 inches; the middle part is 5 feet 8 inches in diameter for a length of 22 feet. The ends are egg-shaped, the thickness of the sheet-iron is 3/16 inch, and the sheets are rivetted together with ½ inch rivets spaced from two to 1½ inches apart from centre to centre. Each pontoon has a manhole, and a small hole for the mooring chain; the mooring chains vary, but are in general long-linked chains ½ inch diameter. No anchors are used, their place being supplied with blocks of stone.

The pontoons are spaced 12 feet apart. The roadway consists of five main timbers *b*, *b'*, 12 inches deep by 9 inches wide, supported, not by saddles, but on two baulks *s*, *s*. (of the same dimension as *b' b*) resting on a trussing *w*, *w*, inside the pontoons. The pontoons are put together in pairs 12 feet apart secured by double beams *b'*, *b'*, thus forming rafts of two pontoons and their connecting roadway: these rafts again are connected by beams *b*, *b*, so bolted as to admit of play at either end. Upon the beams *b*, *b*, etc., *b'*, *b'*, etc., the planking *p*, *p*, consisting of one thickness of 3 inch *sal* is placed. The width of this roadway or planking is 23 feet; it is spiked down to the longitudinal beams at each outside edge of planking. Both above and below runs a longitudinal stringer *l*, *l*, of *sal*, seven inches wide by 2½ inches thick: this is bolted together with wrought-iron bolts; on the top of this, posts *r*, *r*, spaced six feet apart are stepped; between the posts run two lines of long-linked ½ inch chain *c*. Inside the pontoon are placed props of wood to stiffen them (*w*, *w*).

To allow boats to pass up and down, there are two pontoons, each four feet longer than the ordinary ones, on which are arranged two crabs, with hinged platforms. These are raised, the whole removed to one side, and returned when the boats have all passed. The arrangement acts very well.

71. **Raft bridges**—These are usually made of casks or barrels lashed together in lines of six or seven by means of long spars laid across them at the top to act as gunwales. *See* Figs. 41, 42.

Barrel piers are strong and bear grounding well. If in use for any time they must be kept well wetted. The bungs should be uppermost to allow water which may have got in to be pumped out through the bungholes. The commonest cask is the "hogshead" of 54 gallons, which has a safe buoyancy about 500 lb. The span of cask piers is not more then 10' to 12' as a rule. The ends of cask piers should always be rigidly connected to each other by tie baulks lashed to both the gunwales of each pier, and the road-bearers should rest on both gunwales of each pier. Well-lashed piers thus made will last for some months; but they can only be relied on for short periods; they are liable to give way suddenly when the ropes rot which hold them together.

CHAPTER IV

FOUNDATIONS, PIERS AND ABUTMENTS OF BRIDGES

FOUNDATIONS

72. Foundations of large bridges involve some of the most difficult problems that Civil Engineers have to deal with, and require wide practical experience for their successful execution. The nature of the ground to be built upon should first be ascertained by borings or trial pits. When this has been adequately determined, experience, observation, and sometimes experiments are needed to enable the Engineer to choose the kind of foundation, and the method of executing it, best suited to the special conditions of each case.

Foundations may be of three descriptions: (1) Ordinary foundations on dry land, (2) foundations in soil charged with water from which the water must be excluded during construction, and (3) foundations laid under water. Piers and abutments should be carried down if possible to soil sufficiently firm to bear the weight of the superstructure without appreciable settlement, and deep enough to be safe from the scour of rivers or running streams, increased as that action will generally be by the obstacles presented by the bridge piers. If firm soil does not exist for a considerable distance below the surface, the foundations should be carried down to such a depth as will afford the resistance required to carry the load safely by reaching more consolidated strata below and by skin friction.

73. **Ordinary foundations**—For viaducts and bridges built on dry land, ground sufficiently solid to support bridges of moderate height is often found four or five feet below the surface. The piers in such cases are usually made with a slight batter and provided with footings to widen the bottom in proportion to the load and to spread the weight over a large base. Where the ground is unfavourable, the bearing surface is further extended by a wide layer of concrete at the bottom which in very bad cases, may be reinforced with embedded steel work.* The usual rate of pressure allowed in India on ordinary soil, consisting of sand or loam, is 0·8 ton per square foot; on stiff clay this may be increased to 1 ton, or even 1¼ tons, per square foot. On hard canglomerate or soft rock, it may be 1·5 tons to 3 tons per square foot; on hard rock, the limiting pressure is determined by the maximum pressure the

* *See* "Reinforced Concrete" by Buel and Hill, Chapter VI, for a detailed description of reinforced concrete foundations.

material of which the footing is composed will safely bear, which is usually five tons per square foot for brick-work, masonry or concrete in ordinary lime mortar, or eight tons per square foot if Portland cement mortar is used with very hard brick or stone. When the pressure is not uniform, the *maximum* intensity of pressure must be considered in all these cases and not the *mean*.

When the soil is soft and yielding for a considerable depth, bearing piles have often to be resorted to for heavy structures. These, if timber piles, are generally connected together at their heads by a series of walings upon which planking is sometimes laid: or the heads of the piles may be encased in a thick layer of concrete spread over the surface, a system which, besides effectually bonding the piles together, provides a firm level base for the erection of the piers. For important permanent structures, reinforced concrete piles covered by mattresses of the same material are now commonly used. *See* paragraph 99.

74. **Foundations in soil charged with water.**—Frequently, in laying foundations, even on dry land, considerable quantities of water are met with below the surface owing to the excavations being carried below the natural level of the underground water, or to the opening out of springs, or occasionally from percolation from rivers or the sea. In such cases, the water is ordinarily removed during the construction of the foundations by leading it in trenches or pipes to a sumphole dug close by below the owest level of the foundation bed: from this it is raised by a pump and discharged into the nearest natural channel. When the water issues as a spring at a particular spot, it is usually localised by being covered by a vertical pipe or cylinder in which it rises till the height of the column of water in the pipe or cylinder counterbalances the pressure. As soon as the adjacent foundation is completed, the pipe is plugged by cement grout, and the influx is thus permanently stopped.

The rapid influx of water into a foundation trench while it is being pumped is dangerous, as it brings with it the sand silt of the adjacent strata and produces underground cavities liable to cause subsidence above them, which, if occurring under the works themselves or under coffer dams or embankments, may entail very serious damage. It is therefore essential for the safety of a work executed in water-bearing strata that any continued influx of sand or silt, with the water from springs or any other source, should be stopped. This may be done either by sinking a well below the foundation level, beyond but near the side of work, and pumping from it without disturbing the sand as pointed out in the preceding paragraph or by surrounding the side by sheet piles driven down to a firm stratum, after which the water can be pumped out of the enclosure and the sand removed to any requisite depth. *See* pragraph 95 below for a description of piles. In the case of a small bridge, when the current is slow and the obstruction of waterway moderate, another plan for dealing with a sandy bottom, charged with water, overlaying firm soil 10 feet or so below the bottom, is to support the bridge on boxed foundations. These are formed by making large boxes of wood of the shape of the pier, but slightly larger, without top or bottom, and 6 to 10 feet in height as may be necessary. These boxes are driven through the sand into the firm substratum by scooping from the interior and loading them. They are then pumped out and filled with concrete or rubble masonry on which the piers are built. If the ingress of water into the box is excessive and troublesome while the concrete or masonry is being put in, the former may be lowered in skips for the lowest layers, and the latter built in large blocks above ground and when dry, let down and joined below. The above methods are only suitable for foundations 10 to 12 feet deep below water level. Below this depth it is better to resort to well-sinking which is described further on.

75. **Congelation of the soil** by freezing liquids has been successfully employed in England to carry excavations for foundations through running quicksand to a firmer stratum. A series of vertical pipes is sunk into the quicksand round the site, and a cold unfreezable liquid is introduced into these pipes and kept in circulation. In a recent modification, described in " The Colliery Guardian " of 1st December, 1893, page 960, liquid ammonia alone has been used which, in evaporating, produces an intense cold by absorbing latent heat in becoming gaseous. When the site is thus congealed, the excavations can be carried through the hardened soil without pumping or timbering the sides. This system has been found to be very efficient, but it is expensive and would not therefore be suitable in India except under very special conditions which make other methods inapplicable.

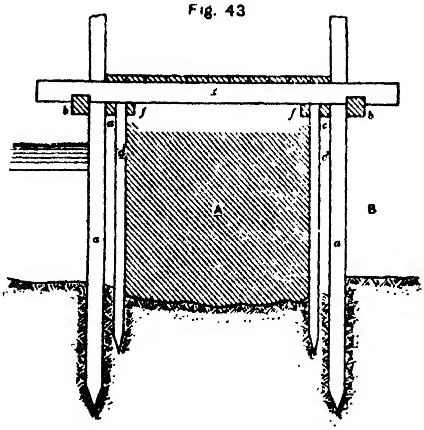
76. **Foundations under water**—In stagnant shallow water, if the bed on which the foundation is to be built is clay or some other firm soil, the water may be excluded from the area on which the foundation will rest by surrounding the area with an ordinary water-tight dam of clay or some other binding earth. It may be necessary to remove some soft or loose soil overlying the firm bottom before the dam is

commenced. The dam is made by throwing in successive layers of clay. When the dam is completed the water is pumped out from the enclosed area and bed for the foundation is prepared as on dry land.

77. When the depth of stagnant water is considerable, say 10′ to 15′ and in *running* water of any depth the ordinary dam must be replaced by the *coffer dam* which consists chiefly of two rows of planks, or sheet piles, driven vertically into the soil, with clay or binding earth between to form a water-tight wall to exclude water from the area enclosed. The arrangement, construction, and dimensions of coffer dams depend on their specific object, the depth of water, and the nature of the sub-soil on which the coffer dam rests.

The width or thickness of the coffer dam, by which is understood the distance between the sheet piles, should be sufficient not only to be impermeable to water but to form, by the weight of the puddling in combination with the resistance of the timber work, a wall of sufficient strength to resist the horizontal pressure of the water on the exterior when the interior space is scooped out and pumped dry. The resistance offered by the weight of the puddling to the pressure of water can be easily calculated; that offered by the timber work will depend upon the manner in which the framing is arranged and the means taken to stay or buttress the dam from the enclosed space. For extraordinary depths the Engineer would act imprudently were he to neglect to verify by calculation the equilibrium between the pressure and the resistance; but, for ordinary depths of 10′ or under, a rule followed is to make the thickness of the dam 10′; and for depths over 10′ to give an additional thickness of 1′ for every additional depth of 3′. This rule will give security against filtration through the body of the dam, but the scantling of the coffer work should also be suitably increased in dimensions according to the height of the dam.

Fig. 43



The usual construction of a coffer dam is shown in Fig. 43 which is a section of one wall. It consists of a row of ordinary stout piles (a) around the area (B) to be enclosed, placed about 4 feet as under. A second row (a) is driven parallel to the first, the respective piles being the same distance apart; the distance *a a′* between the centre lines of the two rows being so regulated as to leave the requisite thickness between the sheeting piles for the clay, etc., forming the dam. The piles of *each* row are connected by a horizontal beam of square timber termed a *string* or *wale piece b b′*, placed a foot or two above the highest water line, and notched and bolted to each pile. The string piece (*b′*) of the inner row of piles is placed on the side next to the area enclosed, and those *b* of the outer row on the outside. Cross-beams of square timber connect the string pieces of the two rows, upon which they are notched, serving both to prevent the rows of piles from spreading from the pressure that may be thrown on them, and as a joisting for the scaffolding. On the opposite sides of the rows, interior string pieces *c*, *c*, are placed, about the same level with the exterior, for the purpose of serving both as guides and supports for the sheeting piles. The sheeting piles *d*, *d*, being well joined, are driven in juxtaposition and against the interior string pieces. A third course of string or *ribbon* pieces *f*, *f*, of smaller scantling, confine the sheeting piles by means of large spikes against the interior string pieces. For a dam about 10′ high from water level to firm bed, the piles *a*, *a*, should be about 9 inches square and the sheet piles *d*, *d*, 9 inches by 4 inches. The square piles should penetrate at least 6 to 7 feet into the firm soil.

The main inconvenience met with in coffer dams arises from the difficulty of preventing leakage under the dam. In all cases the piles must be driven into a firm stratum, and the sheeting piles should equally have a firm footing in a tenacious compact sub-stratum. When an excavation is requisite in the interior to uncover the sub-soil on which the bed of the foundations is to be laid, the sheeting piles should be driven at least as deep as this point, and somewhat below it if the resistance offered to the driving does not prevent it. The puddling A should be formed of a mixture of tenacious clay and sand, as this mixture settles better than pure clay alone. Before placing the puddling, all the soft mud and loose soil between the sheeting piles should be carefully extracted; the puddling should be placed and compressed in layers, care being taken to agitate the water as little as practicable. With proper care, coffer dams may be used for foundations in considerable depths of water provided a water-tight bottom can be found for the puddling.

The method of driving piles is described further on in this chapter.

78. If the depth of water is very great, or when, from the premeability of the soil at the bottom, it is difficult to prevent leakage a coffer dam is a less economical method of laying foundations than the Caisson or the Well. The former may be used by itself in deep water if the bed is rock or some other firm material on which the foundation can be directly placed; but if a very thick layer of sand or silt has to be traversed before reaching a firm water-tight stratum, or where the soil consists entirely of sand or silt for indefinite depths, well foundations, with or without caissons, are the most suitable for heavy bridges, and screw or disc piles for light ones. These will now be described.

79. A Caisson is a strong water-tight vessel of wood or iron, which is made with a bottom to float it but without a top. When a firm bed is available immediately below the water, for construction of the foundation, it is prepared to receive the bottom of the Caisson by levelling the area on which the Caisson is to rest. It is essential that the bed shall be level, otherwise the cross strain on the unsupported parts of the bottom leads to fractures and dangerous movements in the superstructure. In hard ground which is lumpy and cannot be previously levelled all over, the ground should be roughly levelled under the edge of the Caisson only, if uneven, and a Caisson should be used of which the bottom is only an outside frame with a loose tarpaulin fastened to it which can adapt itself freely to the form of the bed. A thick layer of concrete is then laid on the tarpaulin to fill up all inequalities and provide a level bed for the masonry or brickwork of the foundation. The loose tarpaulin is necessary to protect the concrete from the washing action of the water before it has had time to set. If the bed is shelving rock which cannot be adjusted in the above manner, the only way to deal with it is by compressed air as described later under well foundations. In ordinary practice the Caisson is placed in position in the following manner. It is floated to, and moored over, the position it is to occupy. A small sliding gate in the side of the Caisson is then opened to fill the Caisson gradually with water and sink it. When it has been sunk the bed is examined to see if it has been accurately levelled. The gate is then closed and the water pumped out. The Caisson is then refloated The bed is finally adjusted and levelled if this is found necessary. The masonry or concrete of the foundation is then commenced on the bottom of the Caisson and carried up until its weight grounds the Caisson.

80. We come now to foundations in sandy or soft soils liable to scour, where no firm beds can be reached for great depths below the surface. Before describing the methods adopted for constructing

foundations in such places it is necessary to examine the well-known controversy regarding the comparative merits of (1) deep well foundations without floor protection and (2) shallow wells with continuous flooring under the bridge, protected both up-stream and down-stream by piles or rows of contiguous curtain wells.

81. It is argued in favour of deep foundations that it is unwise to interfere with the regimen of a large river whose whole bed of fine sand and silt for a depth of many feet is in motion during high floods, and which discharges a part of its flow below ordinary bed level; that in such circumstances the curtain wells arrest this discharge, and the moving sand and silt sensibly contract the available waterway, so that this undercurrent must be forced either above or below—in the former case causing a dangerous afflux, in the latter undermining the curtain wells and floors to the imminent risk of the comparatively shallow pier foundations. Furthermore, as regards cost, the deep foundation party urge that the only chance of security in the "floor and curtain arrangement" is in the use of a long apron and the constant annual expenditure of material in maintaining a boulder *talas* for some distance up and down the stream which would (eventually at least) bring the cost up to that of deep foundations. The advocates of the "floor and curtain" system on their side argue that in the "unprotected cylinder" arrangement there is no guarantee that the action of floods may not at any time be concentrated in one channel under any one span, and the bed of the river in that span be scoured out to a greater depth than the wells can with economy be sunk to, so that deep sinking cannot ensure safety, which can best be attained by protecting the bed from erosion and thus preserving the waterway uniform. The additional cost and difficulty of sinking wells to great depths are also urged by the "shallow foundation" advocates. The deep well system is often referred to as the Punjab system and the other the Madras system.

It should be a matter for consideration in each particular case which is the best plan to follow. The deltaic rivers in the south of India, with comparatively slight declivity and low velocity, and with beds of large coarse-grain gravel-like sand, acquire the additional sectional area necessary to discharge their extreme flood waters by raising their surface level and widening their course; and such conditions *indicate* the sufficiency of comparatively shallow foundations and the safe adoption of floorings and curtains to ensure a uniform velocity through all the arches of the bridge. Where the soil of the river-bed is moderately firm and the discharge below dry-weather level is small compared with the flood discharge of the river, the system of paved floors is sound in principle, and admits of shallow foundations to the piers. In the great rivers of the Punjab, however, the material of the beds of which partakes of a semi-fluid nature in times of the flood, the mode in which the additional sectional area is acquired differs from that before described; these rivers, when untrained almost invariably wander in their course and scour their beds to gain the required sectional area, so that their rise and fall is comparatively small. Here it is difficult to preserve a *uniform* waterway and to check the motion of the sandy bed. After many years' experience of the great rivers of the Punjab, Railway Engineers have determined that the proper course with deep foundations is to carry the piers to a great depth, contract the width of the river to the actual flood sectional area by means of guide banks as described in Chapter II, and hold the river in this contracted section. Heavy loose stone pitching thrown about the pier wells is used at times to counteract the severe scouring action which occurs in high floods around the wells, when the water rushes past them and forms whirlpools whose depth rapidly increases with the velocity and duration of the floods. The loose stone, by falling into the hollows as fast as they are formed, diminishes the risk from this action, and removes the scour from the piers to the centre of the span, but such pitching must be added with caution, in order to avoid serious contraction of the flood sectional area.

Where continuous flooring is adopted the curtain wells should go down to the same depth as the pier wells to protect the floor from the action of severe under-currents which tend to wash out the sand between the curtains and undermine the floor. Rectangular blocks are more suitable for curtains than round wells as they more readily admit of the interspaces being filled in with piles. But the main security of the floor, and therefore of the bridge, depends upon the efficient construction and careful maintenance of the *apron* which consists of a heavy pitching above and below the bridge of loose rough stones or cribs filled with masses of brick slag. In violent floods there is danger of holes being scooped out in these positions which may extend beneath the curtain and cause it and the floor to fall in. As this pitching falls in and fills the hollows where the sand is undermined or sucked away, the material of the apron must be constantly renewed.

82. **Well foundations**—These are generally made of iron in Europe and of masonry or brickwork in India. The former are commonly

known as *tubular foundations* and consist essentially of large hollow vertical iron cylinders which, after being sunk to the required depth, are filled with masonry or concrete which forms the real foundation of the pier. Well foundations proper, as recently used in India, are thick hollow cylinders or rectangular blocks of masonry which are sunk as far as necessary and themselves carry the superstructure of the pier. After being sunk they are plugged with sand and covered by a dome or vault to form a platform for the pier. To sink the wells, the ground is generally removed from below by a dredger which can be worked from the top and excavates under water. Deep wells may have to be heavily loaded at the top with rails or sand bags to aid their descent, but with the very thick steinings now in use, their weight is often sufficient to take them down to considerable depths without loading.

The methods of construction and sinking iron cylinders are various and a general description only can be given here. The tubes are usually of cast-iron in section of moderate sizes with internal flanges by which they are bolted together in successive lengths as the depth to which the cylinder is sunk increases, till the bottom is reached with the last length still above water. If the bottom is hard, the lowest length is merely fitted to it; if soft, the cylinder may be sunk into it to the required depth, in which case the bottom edge of the lowest section must ser properly shaped to cut into the soil. The excavation of soil from the bottom to sink a cylinder is usually done by divers or by dredging. If the depth is considerable, the divers have to use diving dresses provided with pneumatic tubes which keep them supplied with fresh air fo breathing while they are at work. Under unfavourable condition where the soil is treacherous or the bottom hard and shelving, the compressed air process described further on has to be resorted to. The bottom ring of a cylinder with a cutting edge at the base is made thicker and shorter than the rest, to meet the special strains to which it is exposed. When the cylinder has been sunk to the required depth, the pier foundation is constructed inside the cylinder by filling the enclosed space with concrete and brickwork.

83. The Charing Cross railway bridge across the Thames at London and the Chitravati bridge on the Madras Railway are good examples of bridges on piers built within cast-iron cylinders. The former is described

in Vol. XXII and the latter in Vol. CIII of the Proceedings of the Institution of Civil Engineers.

Fig. 44.
CHARING CROSS BRIDGE PIER

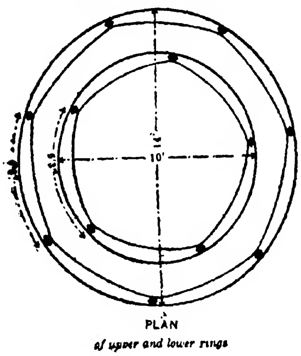
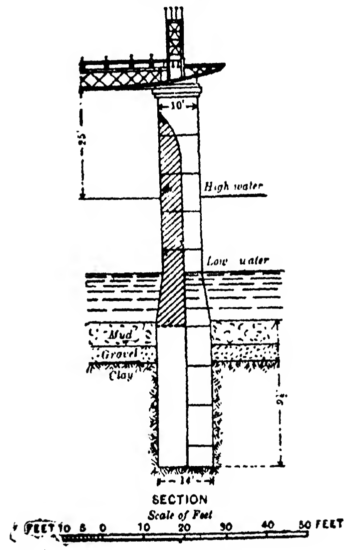
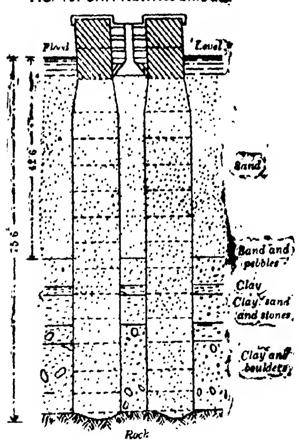


Fig 45
CHARING CROSS BRIDGE PIER



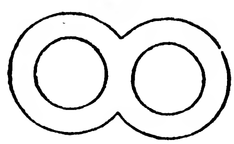
CYLINDERS FOR PIER
FIG. 46. CHITTRAVATI BRIDGE



84. A masonry well foundation may consist of a single well, a twin well or a number of wells sunk close together and connected afterwards

by arches at the top. By a twin well is meant two circular or polygonal wells connected together thus :

Fig 47.



The shape of the wells may be rectangular, circular or octagonal. If the wells are to be pitched midstream, or where water is standing, an island must first be formed by an edging of sand bags filled up inside the enclosure with clay or sand. If they are to be pitched on dry ground an open excavation, a little larger than the well all round, is necessary down to the water table or to the level beyond which the soil cannot stand without support. On the bed thus prepared a curb of wood or iron corresponding in size to the steining of the well is placed and firmly embedded. On the curb the masonry ring or steining of the well is constructed till it is six feet or so above the ground. The sand inside is then scooped out and the cylinder is gradually sunk till its top has very nearly reached the ground surface. Another 8 or 10 feet is then added and again the same process is resorted to. This is repeated till the required depth is attained. If the depth is considerable, it will be found necessary to weight the well with iron rails, or gunny bags filled with sand, supported on a timber platform in which openings are left for the working of dredgers. To secure the curb to the masonry and to prevent the lower part of the cylinder detaching itself from the upper in a sudden descent during sinking, several vertical iron tie rods or bolts, 20′ to 30′ long are built into the masonry of the well from the bottom of the curb, their upper ends being connected under the bolt heads by a 4″ × ½″ horizontal ring.

When the wells have been built and sunk to the full depth, they are filled in with concrete up to a few feet above the top of the curb and then with sand to the top. If sand filling is used, an upper plug of concrete 5′ to 6′ thick is generally provided at the top of the well. When the hearting is finished, the wells are connected together by arches or vaults and the superstructure is built ever them.

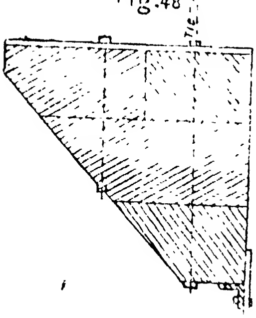
In all cases where the foundation consists of several contiguous wells it is advisable to sink them together, as it is difficult to sink a well satisfactorily close to another already sunk, owing to the disturbance of the soil produced by the latter in going down and the consequent tendency of the former to draw in towards the other and get jammed.

Recent experience has shown that the best shape for foundation wells of bridges is the single circular or octagonal up to 20′ diameter for small piers of short spans up to 60′ or so. For large wells of long span bridges, on rivers thoroughly trained and controlled by guide banks, the twin octagon seems to have given the most satisfaction It is claimed for the latter that it is comparatively cheaper as it corresponds more nearly in shape to the superstructure of a pier, it is easier to sink, easier to keep vertical when sinking, and easier to bring back to the vertical if it happens to get out of plumb during sinking operations. For rivers in sandy soil, with currents constantly varying in direction and scouring the bed to varying depths, some Engineers think it is highly important that the wells of foundations should be cylindrical, even if of large diameter, to present a uniform surface to any direction the current may take and not deflect it as any other shape would do in a greater or less degree according to its section. Rectangular blocks, except for small depths, are now but seldom used.

The thickness of the steining of masonry foundation wells is usually a quarter of the external diameter or, for large wells, an internal diameter large enough to allow of the convenient working of a dredger (say 8′ for 25 cubic feet dredgers) and a steining thick enough to make up the rest with a minimum of 4′ 6″. The depth for moderate spans in sandy soils liable to scour freely is now made by some Engineers 50 feet more than the depth of the maximum bed scour observed in ten mile reaches up and down stream, and by others four times the allowable scour due to the assumed maximum velocity, *see* paragraph 16 (8). The depth for very large spans is governed by considerations other than those of scour; *see* paragaaph 100 on "loads on foundations."

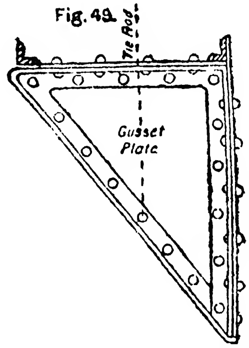
85. Strong and expensive curbs of iron are needed for the large wells of big bridges, but in ordinary cases where sinking is easy and even, and the depth not great, the curbs are usually made of some cheap hard wood readily obtainable in the locality, *e.g.*, keekar, mulberry or sheesham. The wood is cut to shape in sections and arranged in two or more layers in depth as shown in the sketch below. The larger the piece the better, as the joints will be fewer. The pieces are secured together horizontally by dovetail dowels and break joint both horizontally and vertically. They are further connected together transversely, and in depth, by iron straps and bolts, the lowest layer having a cutting edge of half inch plate steel secured to it by means of an angle iron.

Fig. 48



Wooden Curb

Fig. 49



Iron Curb

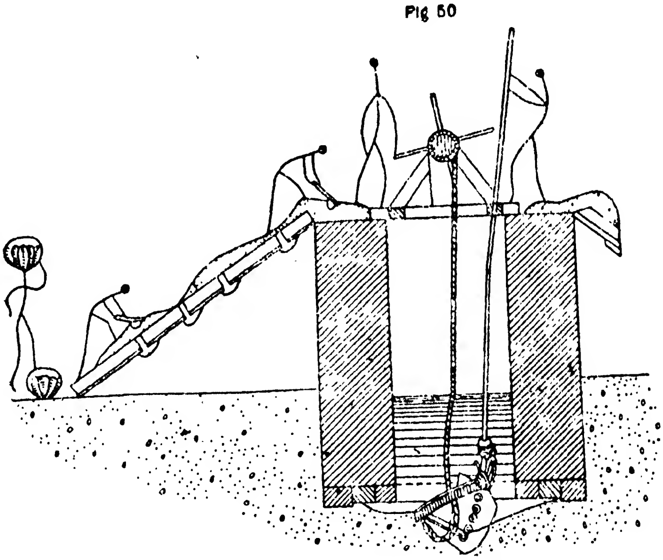
Iron curbs consist of internal triangular ribs of framed angle irons, connected together at intervals by stout angles at the two upper edges and fitted with a cast steel shoe at the lower. These ribs are encased in $\frac{3}{8}''$ to $\frac{1}{2}''$ steel plates. The angle at the shoe should not exceed 30° to facilitate excavation under the curb, if necessary, and to give a deep section for greater strength. *See* Fig. 49.

86. As long as the water in the interior can be kept out by pumping or lifting, the work of sinking the wells proceeds quickly, but when the work has to proceed under water it is very slow and dredgers have to be employed to take out the core of the wells.

87. The *jham* is the original Indian machine used for this purpose. It is the simplest and best for small wells and consists of a huge *phaora* or hoe with a straight socket into which a long pole handle is fitted, by which, when lowered into the water, it is kept upright with its edge downwards and can thus be worked into the sand from the top of the well. This can be done either by pushing with the pole or by blows of a

heavier pole guided by a diver, on to the head of the straight socket. When the *jham* has been pressed home into the soil, the pole is withdrawn and, by means of a windlass and rope attached to the inclined arm above the spade, the *jham* with it load of sand is dragged up and emptied. In some parts of the country, well-sinkers dive down every time and work the *jham* into the sand by their hands In case of serious obstruction to the regular descent of the well, due to boulders or tree trunks, divers are employed to remove it. Fig. 50 shows the working of a *jham* for sinking a small well.

Fig 50



Jham being sunk into soil kept upright by pole

88. For big bridges, which require numerous wells in their foundations, the progress of well sinking would be too slow if *jhams* were used. Various other arrangements have therefore been devised to do the work of excavation more quickly. The machines now generally used in India for this purpose are dredgers. There are many different varieties of dredgers in use. Bull's dredger was one of the first in the field and it is described in detail on page 59 as it is fairly representative of the type. On large works it has been superseded recently by Bruce's digger and Bell's dredger which are much larger and can lift 40 to 50 cubic feet at a time. The diggers employed in sinking the wells of the Dufferin bridge at Benares were 8 feet in diameter and lifted 130 cubic feet

of earth. *See* proceedings I. C. E., Vol. CI. The usual size is the dredger of 25 cubic feet capacity. These very large dredgers are worked by steam hoists and their buckets are designed differently to suit the peculiar soil to be dealt with. Those intended for excavating stiff clay are provided with toothed grabs instead of the plain buckets. A description of most of the dredgers and grabs in common use will be found in Vol. LXXXIX Proceedings Inst. C. E. Bruce's digger is the most suitable for pure sand.

89. The Bull's dredger is shown in Plate V. A short chain 4 feet long with a ring in the centre, should be attached by its ends to the rings on the chains working the machine. To the centre ring the chain for lowering and raising the machine is fixed, of a length greater or less according to the depth of the well. On the well, shear legs are erected with an iron block made fast to the junction. The shear legs should not be less than 10 or 12 feet in length, stayed on either side to the ground. A wooden platform 6 feet × 4 feet, composed of stout planks made fast to two under cross pieces, is laid on the well, and two ¾ inch ropes are attached to the dredger, one made fast to the key keeping the jaws of the machine open and the other to the centre ring in the short chain first mentioned.

In working, the machine is opened on the wooden platform and the key is fixed. It is then lowered into the well, and on reaching the bottom the key is withdrawn. The rope attached to the latter should be coiled on one side of the platform ready for use. A *gentle pulling-and-giving motion* should now be applied with the rope attached to the centre ring of the short chain, *slowly at first,* and *as this peculiar motoin causes the jaw of the machine to sink or cut into the sand, the strain should be increased,* till there is no further yielding to the pull which two men can put on the rope. The machine should then be raised and landed on the wooden platform. The operation of re-setting it, for lowering, releases the sub-soil brought up, and saves all trouble in emptying.

The average quantity brought up, when the machine is properly worked, is 2 cubic feet, and in a well of 12 feet 6 inches diameter, 38 feet deep, there is no difficulty in working it 25 times in an hour. Three men on top of the well (not including those employed in removing the sand, which is best done by contract) and 15 men to pull, are required to work the dredger. The average performance per day in a 12 feet 6 inch well is 3 feet of sinkage, and practically speaking the depth of the well is of no consequence, the difference of time taken by the coolies walking 10 feet or 50 feet being inappreciable as compared with the time taken by each operation. The dredger is intended chiefly for working in sand, but brings up anything which is cut up so that it can grip it.

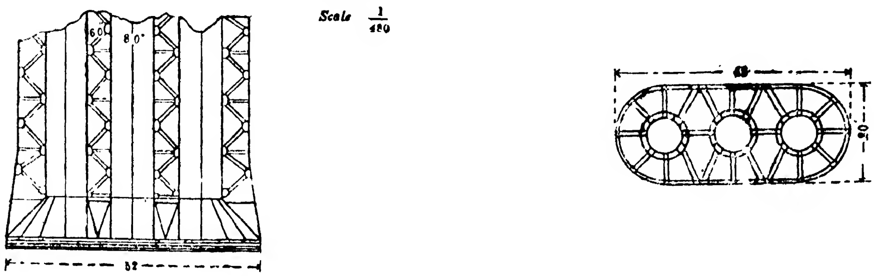
90. **Working of large dredgers**—Bull's dredgers are now used for small wells only. For sinking numbers of large wells for a big bridge, the large diggers or dredgers described in para. 88 are commonly used with steam hoisting machinery and long shear legs, 50′ to 60′ high, which stand clear of the well and allow the sinking and building of fresh lengths to go on uninterruptedly. The short shear legs referred to in the description of a Bull's dredger, which are placed on the well and have to be removed and re-erected every time a length of cylinder is sunk, involve a great waste of time and money. When the sinking operations are on a large scale, the emptying of the large dredgers is also now done automatically and the soil is ejected clear of the well. Up to depths of 60 feet these improved arrangements render the sinking a very simple mattter, but when much greater depths have to be reached, heavy loading becomes necessary and the alternate placing and removal of the loads makes the work more expensive.

91. **Wrought iron caissons for foundations**—Where the foundations have to spread over a large area for the piers and abutments of very large bridges with as little obstruction as possible to the waterway, especially in deep water and soft beds, continuous wrought iron caissons enclosing the whole site of each pier, but separated generally into sections by internal partitions, present advantages over masonry or iron cylinders. Like these cylinders they are provided with a cutting edge round the outside and are sunk by similar means Figs. *51, 52 show the wrought iron Caissons used for the bridge over the Hawkesbury estuary in New South Wales. These caissons have three dredging shafts inside, each 8 feet diameter, strongly braced by diaphragms to the outer shell. The shafts are splayed at the bottom to facilitate dredging, but recent experience is against this practice, as vertical sides have been found to have the advantage of rendering the Caisson less liable to cant over during its descent and easier to get straight if it happens to go out of plumb in sinking. The Caissons were floated into position by the help of a false bottom and gradually sunk, by filling in the spaces between the shafts and outer shell with concrete, through a stratum of mud reaching a thickness of about 120 feet, to a sandy bed below.

*"Civil Engineering" by Vernon Harcourt. By permission of publishers.

The bottom of the foundation of one of the piers was 162 feet below high water of spring tides. When the stratum of sand was reached the shafts were filled with concrete, and the masonry piers were erected upon these foundations from about low water level.

CAISSON FOR PIER
FIG. 51 & 52 HAWKESBURY BRIDGE N.S.W.



SECTION

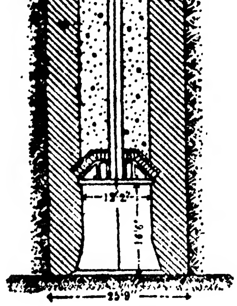
A full description of this bridge will be found in Vol. CI,, Proceedings I. C. E. Two other good examples of wrought iron Caisson foundations, which might be usefully studied by students desiring further information on this subject, are those of the Dufferin bridge over the Ganges at Banaras, and of the railway bridge across the river Hughli, at Hughli, which have been described in Vols. CI and XCII of the Proceedings I. C. E., respectively.

The lowest section of the Caisson, 10′ to 15′ in depth, formed in the shape of a curb at the bottom, is usually built on shore on inclined baulks of timber called "ways" or "slips" which slope lengthwise down into the water at about 1 in 9. During erection, the Caisson is wedged up on these slips. When it is ready to be launched, the slipways are greased and the wedges uniformly and systematically removed to allow the Caisson to slip into the water and float. The slipping process may have to be assisted by a pull from a rope worked by a crab winch on a barge in the river, but this should not be necessary if the slips are properly arranged. Several plans may be adopted for lowering Caissons into place after they have been floated, as well as for the flotation itself. One plan is to make a false bottom about 2 feet up from the cutting edge and sink the Caisson by pumping water into it. Another is to build the Caisson on shore to a certain depth, launch it into the water like a ship until it floats in about 8 to 10 feet of water as described above, and then haul it into position and sink it by building up and concreting. A third method which was adopted at the Dufferin Bridge,* Banaras, and seems to be the most reliable and safest, is to build the Caisson on a staging supported between the two pontoons, one on each side, and lower it gradually by tackle from an overhead staging built on the pontoons across the Caisson. This method secures full control over the lowering into place and is illustrated in Plate VI.

92. **Compressed air foundations**—In sinking cylinders through quicksand of very fine silt, trouble frequently occurs through the sudden influx of the surrounding material, and difficulties are experienced when foundations have to be laid on a shelving bed of rock which requires cutting at a considerable depth below surface, or when the cutting edge comes in contact frequently with large boulders or trunks of trees. In these circumstances, by the use of compressed air, sand and silt can be kept out, the cutting of the rocky bed into steps or flats can be executed in the usual manner, and the removal of boulders or other obstructions under the cutting edge can be dealt with easily. Moreover, the concreting can be done in the dry, and there is no necessity to lower it in skips and deposit it under water as is usually done when wells are sunk in the ordinary way without the aid of compressed air.

The essential feature of the compressed air system is the bottomless drum forming the lowest portion, 6 to 8 feet high This is the working chamber filled with compressed air in which the workmen excavate the material from the bottom and thus gradually cause the cylinder to descend till it attains the required depth, when it is filled with concrete. The roof of the working chamber is made very strong to support the structure built on it up to water level while sinkin is in progress. The cylinders sunk in this manner are generally of wrought iron, but they have sometimes been made of wood as in the piers of Brooklyn Bridge at New York, and occasionally of masonry as in the dock walls at Rochefort shown in Fig. 53. When masonry cylinders are used the roof is of vaulted masonry.

FIG. 53. ROCHEFORT DOCK WALL.



* See Proc. Inst. of Civil Engineers, Vol. CI.
† Civil Engineering by Vernon Harcourt. By permission.

The working chamber is entered through one or more airtight vertical shafts constructed over openings in the roof, each provided with an air lock at the top, for passage of men and materials between the open air above and the compressed air in the working chamber below. These air locks usually consist of cast iron boxes with two doors, one a trap door at the top opening downwards from the external air, and, at one side or below, another door opening towards the interior of the passage. Each lock chamber is provided with stop cocks communicating with the external air and the interior of the passage respectively which can be opened and closed by persons either within the passage, within the box, or outside of both. The lock chamber is further provided with a valve and a pipe for introducing compressed air into the working chamber and passage, a pressure gauge, and a large escape valve for releasing the compressed air suddenly when required. Air is pumped into the passage and working chamber, by steam power till all the water is expelled and the bottom on which the cylinder stands is exposed under the compartment. The work is done as follows. The upper door being open, the workmen enter the lock chamber and shut it; then the lower door stop cock is *gradually* opened and admits the compressed air from below till the pressure is even on both sides of the door. The door can then be opened. The men are in the compressed air and free to descend to the compartment. To get out, the process is reversed, a chamber full of compressed air being lost at each opening. The men work at the bottom, dry and comfortable, but under pressure of the compressed air which is trying at great depths. To work with safety under compressed air the men should be healthy and temperate, and the air should be frequently renewed and kept fresh. The insensibility which sometimes seizes a man on emerging direct from a high pressure into the open air, and is sometimes attended with dangerous results, can usually be cured by placing the sufferer in a chamber of pure compressed air at lower pressure which is *slowly* run down to the normal atmospheric pressure. The maximum pressure which workmen can endure comfortably for long periods is *35 lb. per square inch which corresponds to about 80 feet depth of water.

In order to reduce the cost of sinking by compressed air for foundations in shallow water, the working chamber Caissons have been made removable in some cases, being raised by screw jacks as the masonry is built up. By this means the Caisson forming the working chamber can be used over again for successive foundations instead of remaining

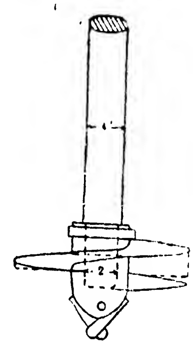
*See Proceeding Inst. C. E., Vol. CXLIV, pp. 48 and 100.

embeded in the masonry which enhances the cost of the foundations in shallow water very considerably. *

93. **Screw pile foundations**—Where a light bridge or viaduct with moderate spans has to be carried across a river or creek with an alluvial bed of considerable depth, iron screw piles have been used with advantage. These piles are made up of lengths with flange joints bolted together and can be of any required height with proper bracing. They present but little obstruction in proportion to their strength, and, when braced together in clusters, they form a rigid pier. They cause very little disturbance of the ground in being screwed down by long bars at the top and the projection of the screw increases considerably the bearing surface of the pile on the soft bed.

The screw blade, which is fixed to the foot of the pile is usually of cast-iron, and seldom makes more than a single turn round the shaft. Its diameter is from twice to eight times that of the shaft of the pile and its pitch from one-half to one-fourth of its diameter. The best mode of driving screw piles is to apply the power of men or of animals, walking on a temporary platform, directly to levers radiating from the heads of the piles.

Fig 54



As an example may be cited the wrought-iron piles used in the piers of Railway bridges on the Bombay and Baroda Railway. Each of these was screwed into the ground by means of four levers, each 40 feet long and each having eight bullocks yoked to it. In this example, the greatest working load upon each screw of 4 feet 6 inches in diameter, *exclusive* of the earth and water above it, is nearly as follows :—

Pier 25 tons + superstructure 12 + train 30 = 67 tons = 150,080 lb., being at the rate of nearly 100 lb. per square inch or 7 tons per square foot of the horizontal projection of the screw blade.

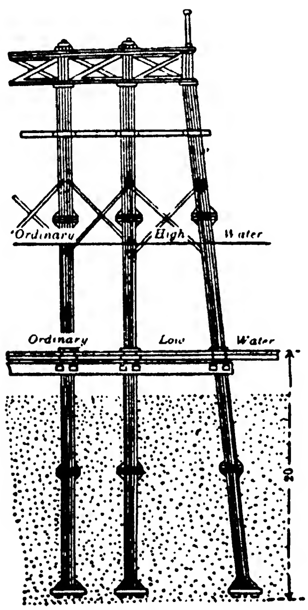
94. Very similar to screw piles in principle are hollow iron piles with a disc at the bottom. *See* Fig. 55. These are usually of cast-iron and are sunk by a stream of water under pressure from a pipe lowered inside the pile which scours away the ground from under the disc and causes the pile to descend. They are sunk very readily through pure sand by this process, but when the soil is more compact owing to the presence of

* See Annales des Ponts et Chaussees 1,839 (2) p. 461 and plates 50 54.

the presence of clay or silt, the progress of sinking is expedited by providing the bottom of the disc with sharp radial ribs as in Figs. 56, 57 and by turning the pile slightly as it sinks.

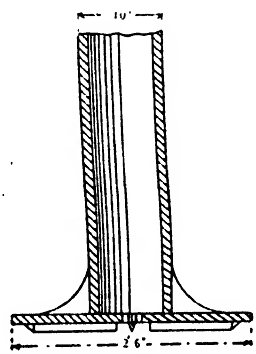
When the sinking has been completed by the water jet, a few blows on the pile from a heavy ram with a small fall give the disc a firm solid bearing on the sand which might previously have been disturbed somewhat by the water jet.

Fig 55



Scale $\frac{1}{192}$

Fig 56



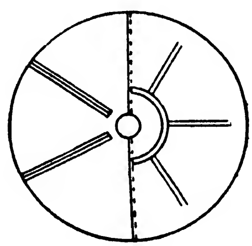


Fig 57.

Scale $\frac{1}{10}$

95. **Piles and pile driving**—The use of ordinary driven piles in foundations has been referred to in previous paragraphs. They will now be described, as also the method of driving them into the ground.

Timber piles are generally 9 to 18 inches in diameter with a length not above 20 times the diameter. For important works they are of sawn timber, but if sapling or rough timber, is used they should be prepared for driving by stripping off their bark and pearing down the knots to reduce as much as possible the friction in driving. The head of the pile is usually encircled by a strong hoop of wrought iron to prevent it from splitting by the action of the ram. The lower end is pointed and protected by an iron-shoe fastened to the side of the pile by straps and pikes.

96. A machine termed a pile Engine is used for driving piles. It consists essentially of two uprights firmly connected at the top by a cross piece, and of a *ram* or *monkey* of cast-iron, for driving the pile by percussion. Two kinds of engines are in use; the one termed a *Crab Engine*, is shown in Plate VII; in this a heavy monkey is hoisted by a crab winch to the full height and released by the detaching arrangement shown in detail. In the other, the *Ringing Engine*, a light monkey is raised by the sudden pull of several men upon a rope, or ropes, by which it is drawn up a few feet to descend on the pile, thus giving a much lighter blow than the other, but the blows for more quickly in succession. *See* Plate VII. In ordinary cases the ram is raised by manual labour, but for heavy work which must be quickly done a steam hoist is used which works an endless chain provided with catches at intervals to raise the ram, or a steam pile driver may be used like the well-known Nasmyth's machine, which consists of a heavy iron frame resting on the pile and forming the steam cylinder, with a hammer or ram inside it constituting the piston. The latter is very efficient in sandy soils as it works a very heavy ram (4 tons) dealing 60 to 80 blows a minute with a fall of 3 to 4 feet.

The fall given to the ram varies from a few feet up to a maximum of 30 feet, and the weight of the ram from one-third of a ton to two tons, according to circumstances.

In calculating the effect of a ram in driving a pile which does not rest on a firm bed but resists only by virtue of friction on its sides arising from compression of the soil, French Engineers have adopted a rule to stop the driving when the pile has arrived at its *absolute stoppage*, which is supposed to be reached when a volley of thirty blows from a ram of 800 lb. falling 5 feet does not produce a further penetration into the subsoil than $\frac{1}{5}$ of an inch. Another rule (known as Sander's rule) is that the safe permanent load a pile will carry is $\frac{W.H.}{8\ D.}$ where W is the weight of the ram, H is the fall of the ram is inches and D the set of the pile with the last blow, also in inches. Many other rules of this kind will be found in Molesworth's and Trautwine's Books of Engineering Formulae, but Sander's rule given above seems to be as good as any.

A heavy ram with a small fall is the best for sand; a light ram with a high fall for clay. A great number of light blows in quick succession is more effective than a small number of heavy blows especially in sand.

When a pile, from breaking or any other cause, has to be drawn out, it is pulled up by using a long beam as a lever for the purpose, he pile being attached to the lever by a chain or rope suitably adjusted.

If this is not feasible in deep water, hydraulic or screw jacks carried on a pair of boats or pontoons may be used. It should, however, be noted that the drawing of piles often causes a settlement of the ground which may be dangerous, and where this must be avoided the piles should be left in the ground, their projections above ground being cut off as far as may be necessary.

The least distance apart at which piles can be driven is about $2\frac{1}{2}$ feet from centre to centre. If they are more crowded than this they may force one another up as they are successively driven. It is therefore a matter for calculation whether the weight to be borne can be supported on the area available.

As an aid to pile driving, water jets have sometimes been employed, where pure sand or soft soil has to be dealt with, in the same manner as that adopted in the case of hollow iron disc piles described above. By placing two flexible pipes down opposite sides of a pile with their nozzles extending below the pile and directed centrally towards its point, and turning on a powerful jet of water under pressure through each, the soil under the pipe is greatly loosened and scoured and the pile sinks much more freely with far less driving.

97. **Sheet piles** are flat piles which, being driven successively edge to edge, form a vertical sheet for the purpose of excluding soil and water from foundation trenches during construction, or preventing the materials of a foundation from spreading. They may be of any thickness from $2\frac{1}{2}$ to 10 inches according to the depth of soil they hold back. They are generally bevelled at the lower end to an edge and in stony ground may be shod with sheet iron. They should be carefully fitted to each other before driving. Their edges are sometimes tongued and groved, but this is seldom necessary; if they are driven perfectly parallel and true the swelling of the wood when exposed to moisture will generally secure a watertight joint. When the piles are thin, and their heads are liable to split under the blow of the ram, a timber capping piece is placed on top of the pile to receive the direct blows of the ram and transmit them to the pile. Steel sheet piles are now frequently used.

98. **Iron and Steel piles**—Wooden piles are liable to rot quickly in India if they are in positions where they are alternately wet and dry. Metal piles have therefore been much used in England and India where wooden piles would not be suitable, and they have been found particularly useful in the deep sandy beds of Indian rivers. They are generally of cast iron, but frequently of steel, and may be used either as bearing piles or sheeting piles. The former may be either solid of section ✢ or hollow

(square or round). The fall of the ram should not exceed 4 or 5 feet as cast iron is brittle and would be fractured under a higher fall. A piece of hard wood (called a *punch* or *dolly*) should always be interposed between the ram and the pile, in driving, to deaden the blow. Cast-iron piling should not be used in *salt* water as it becomes gradually softened after a time so that it can be cut with a knife.

99. **Reinforced concrete piles** have recently been used in the foundations of large work in Europe and America. The high cost of Portland cement and steel as compared with timber will preclude their use in India except in very special cases. They are used in the same way as timber piles, and in construction they are not essentially different from columns of armoured concrete. Their chief advantages are (1) they are equally durable in dry or wet soil where piles of timber must be kept constantly submerged to reserve them from rotting, and (2) they are particularly suitable for use in sea-water in which cast-iron piles are useless. Further information, if required, can be obtained by reference to the standard books on "Reinforced Concrete" by Marsh and Dunn or Buel and Hill.

100. **Loads on foundations of piers of bridges**—The weight on bridge supports increases necessarily with the span and height of the piers. For shallow foundations the safe intensities of pressure have been given in paragraph 73. For piers with deep foundations provision has to be made to carry the load by (1) giving the foundations as large a base as possible (2) carrying it down to such a depth as will afford the required resistance by (*a*) the greater compactness of starta in the lower depths, and (*b*) the friction between the soil and the external surface of the foundation imbedded therein. If the pier stands in deep water the effect of buoyancy must also be considered as aiding the resistance of the soil.

As regards the splaying of the foundation to give a larger base, this can only be done to a limited extent in deep pier foundations owing to difficulties of sinking and of obstruction of waterway. The resistance offered by compact strata at great depths may be calculated by Rankine's formula for deep foundations in Art. 196 of his Applied Mechanics. According to this formula, the greatest intensity of pressure, consistent with stability, of a structure founded on a horizontal stratum of earth at a depth X, the angle of repose being γ, is $W X \left(\frac{1 + \sin \gamma}{1 - \sin \gamma}\right)^2$, W being the weight of a cubic foot of earth. The angle of repose may be taken to be 38° for dry sand, 22° for ordinary wet sand, 15° for running sand, 4° for dry clay and 16° for wet clay.

To calculate skin friction, the co-efficient of friction between masonry or brickwork and clean wet sand may be taken to be 0·4. An actual experiment made in India with a well 19′ 6″ diameter, sunk 40′ below ground surface, showed that the skin friction per square foot amounted to 5 cwt. per square foot. See Vol. CLI. Proceedings I. C. E., page 297. This however seems excessive. A safe allowance for wells in sand 60′ to 70′ deep would be 3 cwt. per square foot. For greater depths up to 150 feet it may be increased gradually to 5 cwt. and for smaller depths up to 25′ reduced to 2 cwt. See Molesworth's Pocket Book of Engineering Formulae, page 89, 26th edition.

Buoyancy is easily calculated from the water displaced and will of course depend on the depth of immersion.

101. The Hugli bridge piers, sunk through 30 feet of water and 60 feet of silt, rest on yellow clay. Deducting skin friction at 5 cwt. per square foot, the weight on the base of the pier is 7·5 tons per sq. foot, the normal pressure of earth and water at the same depth being 4·5 tons per square foot. The Dufferin bridge piers at Benares rest on sand at a depth of 140′ below low water level, of which 60′ is water and 80′ sandy soil; excluding skin friction and buoyancy, the weight on the base is 11·19 tons per square foot. The Hawkesbury bridge piers in New South Wales have been sunk through a stratum of mud 120 feet thick to a sandy bed below; excluding skin friction and buoyancy, the weight on the base is 9 tons per square foot.

102. **Piers and Abutments**—The design of a bridge depends primarily on the spans selected for its openings, and the spans again are determined by the number and position of the piers. When the piers are of small height and their foundations easy, small spans and numerous piers furnish the most economical form of bridge. When, on the contrary, the piers have to be made a considerable height, and more especially when the foundations have to be carried down a considerable depth below the bed of the river, it is usually necessary on the grounds of economy to adopt large spans in order to reduce the number of piers. See paragraph 36, Chapter II.

103. As the horizontal thrust of the two half arches on each side of a pier counteract each other, the only force that the pier of an *arched* bridge has to resist is that due to the weight of the two half arches and their superincumbent load. In a *girder* bridge the load is also vertical and arises from the weight of the two half spans on either side and their moving load. The thickness of the piers at the top to sustain this

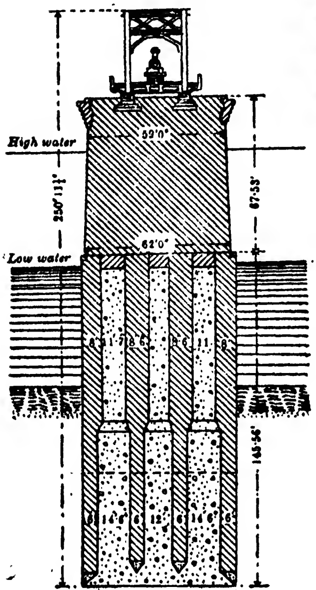
vertical pressure, if calculated theoretically, would be very small as the crushing stress of brickwork is 35 to 40 tons per square foot, but this is much less than what is given in practice, for allowance must be made for the wear of the exterior surface in the course of time, for shocks due to earthquakes and floating bodies, and for the size of bearing plates necessary for the size of girders to be used. The safe intensity of pressure on the soil on which the foundations rest must also be considered. The ordinary dimensions for brick or masonry piers of arch bridges are one-sixth of the span for spans from 15 to 30 feet, one-seventh from 30′ to 60′, and one-eight to one-tenth for larger spans, the proportion getting smaller with the increase of span. This thickness is measured at the top of the pier. The usual widths of piers of railway girder bridges are as follows—3′ for spans of 20 feet, 4′ 6″ for spans of 30 feet, 6′ for spans of 40 feet, 7′ for spans of 60 feet, 9′ for spans of 80′, and 10′ for spans of 100 feet. Short piers may be built with perpendicular sides, but a very high pier looks better with a batter of 1 in 12. Piers for girders and semi-circular or semi-elliptical arches are finished off with flat tops or imposts, but, for segmental arches, inclined skewbacks are provided at the top of the piers. The upstream face is built with a cut water formed thus  to direct the current, with as little eddy as possible, through the openings between the piers.

104. In bridges composed of numerous arches, it is advisable to make every fifth or sixth pier strong enough to act as an abutment, as the arches can then be turned in sets and great expense in centering is thus saved. Moreover, in the case of injury to the bridge by floods causing the fall of an arch, the damage will probably not extend further than the next abutment pier.

105. Sections of the piers of the Dufferin railway bridge, Benares, which carry spans of 356 feet are given on opposite page in Figs. 58, 59. They are typical of the construction of piers of large bridges on very deep foundations.

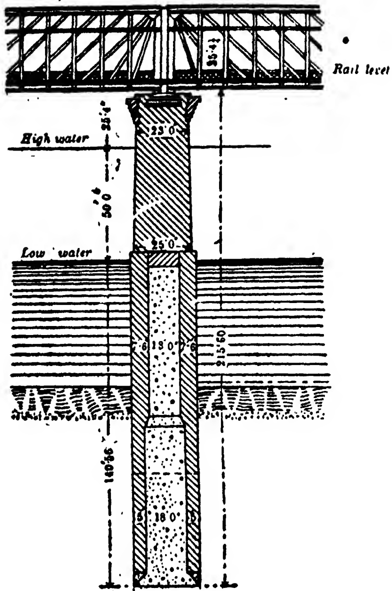
106. Abutments for girder bridges should be thick enough to take the vertical load due to the weight of half the span with its moving load, and also to act as retaining wall for the filling behind the abutment when the abutment *is unloaded* (as it will be before the erection of the girders and perhaps at some future time during their renewal). Abutments of arched bridges having to sustain the thrust of the end arches, unbalanced by the counterthrust of any adjoining one, require to be thicker than piers; and as the thrust depends on the span, rise and weight of the loaded arch, this should be calculated in each particular case and the thickness deduced therefrom as explained in the chapter on arched

Fig. 58.



LONGITUDINAL SECTION.

Fig. 59.



CROSS SECTION.

bridges, paragraphs 115, 116. It should, however, be remembered in connection with this that if the wing walls and buttresses of an abutment are built up and the filling of earth behind it is rammed in and carefully executed simultaneously with the building of the main abutment wall, the whole mass of masonry and filling behind the abutment will increase its resistance to the thrust of the arch which is eventually to come on it. The following are good forms of abutments in connection with wing walls.

FIG. 60.
No 1

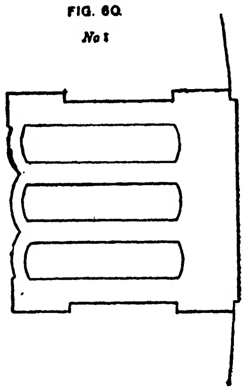
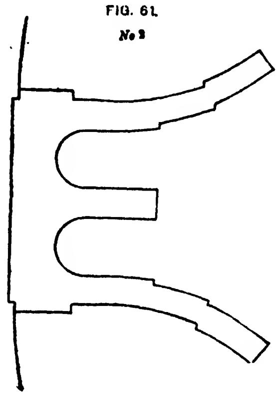


FIG. 61.
No 2



107. **Wing-walls** may either be built at right angles to the abutment, or with a splay outwards; the first plan is the simplest, and has the advantage of giving a better hold to the earth of the river banks; but when the road narrows on crossing the bridge, or when two or more roads meet at the approach to it, the wings require to be curved or splayed. When the soil of the river banks is bad, the foundations of the wings must be laid at the same depth as that of the abutments for their whole length, otherwise unequal settlements and cracks are likely to result from the unequal height and weight of different portions. But if the soil forming the banks of a stream be firm, the wing-walls may be cheaply built with foundations in steps corresponding with the form of the banks. This procedure is not recommended, however, except for very compact soils or rock. The depth of each successive step should be determined with reference to the section of the bank and to the soil; in rocky ground the surface of each step should be 2 feet below the surface of the ground, and in gravel 3 feet. When the foundations are stepped, lengths of different heights should not be bonded together but built with clear vertical butt joints, otherwise cracks are likely to appear from unequal settlement.

A good rule for the length of wing-walls is, that they should be one-and-a-half times as long as the height of the road-way above the bed of the river. Their mean thickness may be one-fourth of their height, and they should be built with offsets on the inside so as to reduce the thickness at the top to 2 or $2\frac{1}{2}$ feet. This thickness may be reduced in the case of long wing-walls, partly buried in filling on both sides, if the filling is done simultaneously with care inside and out. In fact the rules for their thickness are the same as for other retaining walls. The thickness of wing-walls should depend not only on their height but on the description of the soil; when the soil is firm and compact there will not be so great a pressure on the wings as when it is loose. Wing-walls of very great thickness have been thrown down or cracked to such an extent that it has been necessary to rebuild them, owing to the earth having been thrown in loose between them and swelling after becoming saturated with water. This accident would never happen if the precaution were taken of filling in the earth *gradually* as the walls were built; it is then trodden on daily and cannot afterwards be easily penetrated by water. When abutment walls are built with long buttresses] the pressure on the wing-walls is decreased.

The ends of wing-walls are generally widened to form pillars of support which may be square below ground, and of any form above ground which may be considered most appropriate to the design of the parapets to which they form terminations.

CHAPTER V

ARCHED BRIDGES OF MASONRY, BRICKWORK AND CONCRETE

108. Arches of brickwork, masonry and concrete are very commonly used for road bridges of moderate span where the available headway is adequate, on account of their simplicity, durability, and when materials are readily available, their moderate cost. They are sometimes used for railway bridges.

109. **Shape of arches of bridges**--The arches may be semi-circular, elliptical or segmental. Semi-circular arches are strong and have the advantage of exerting little or no thrust on the abutments, but they generally give too great, a rise, thereby involving heavy and expensive approaches. Elliptical arches are very light and graceful in appearance and exert but little thrust on the abutments, but they are not so strong as the segmental type and are more difficult to construct. The segmental arch is the type usually adopted. The proportion of rise to span ordinarily varies from one-seventh to one-fourth, the ratio increasing with the span. The flatter an arch is the thicker it must be to resist the compressive strains it has to bear and the greater the care required in its construction ; the higher it is the more expensive are the approaches, if in embankment.

The springing of the arch should be a little above the level of the highest known, flood, and, where navigable rivers or canals are dealt with, sufficient headway should be allowed to give room for a laden boat passing under the arch at the high water level.

It is now usual to give the same rise and span to all the arches of a bridge and to place their springing lines on the same level. This is preferable to the older practice of having arches of unequal dimensions with their springing lines lower in receding from the centre to the extreme arches. When the first plan is adopted, the parapet wall and roadway are on the same level throughout, and measures must be taken for keeping the latter dry by apertures at the bottom of the parapets. The latter plan has, however, the advantage of reducing the cost of the approaches by having a gradual fall in the roadway from the crown to the two ends, and many still think it is more picturesque than the other.

110. **Bonds of arches and other minor details of construction**—The bond of arches and other minor details of their construction have been dealt with fully in the Manual on Masonry. The wooden centerings, required to support the arch while it is being built, have been

described in the Manual on Carpentry, but there are certain points connected with the centerings peculiar to the construction of large bridges, which must find a place in this Manual.

The actual working period in Upper India for fixing the centerings and turning all the arches between two abutments or abutment piers is ordinarly nine months, *viz.*, from 1st October to the 30th June. On or about the 1st July the monsoon rains begin, and high floods may occur at any time from that date to the end of September. It is, therefore, advisable that all centerings, etc., obstructing the waterway should be removed by the 1st July and the spans left cléar for the passage of flood water. Under these conditions it is most essential that all preliminary arrangements for building materials, pile driving apparatus and labour should be complete by 1st October to enable a good start to be made without loss of time as soon as the flood season is over.

111. **Arch centerings**—In bridges over large rivers, dry weather stream for the greater part of the year passes through one or two spans only, for which open wooden centerings, either spanning the whole opening or built on intermediate pile supports, are absolutely necessary. These will be designed as explained in the Carpentry Manual. For the side spans, which are dry except in the rains, it may be possible in bridges of moderate height to make solid centres of sand taken from the bed of the river. Where woodwork is expensive and coolie labour is plentiful these solid centerings will sometimes be found to be more economical and more quickly made and removed than timber trusses and supports. In making solid centerings care should be taken to see that the filling behind the piers adjoining the open spans is not carried to such a height as to cause any lateral movement of the pier. If calculations show that this is likely, the filling should only be taken up to the safe limit, and the rest of the space up to the arch built up with substantial temporary pillars of bricks on broad bases resting on the filling. Such a combination of filling and temporary brick pillars is also useful when the arches have a considerable rise, which would necessitate heavy earthwork in the end slopes of the filling if carried right up to the crown. The filling in such cases may be carried up to the springing, and the segment built up with pillars to carry the arch during construction. To prevent any appreciable settlement while the arch is being built, the sand filling should be watered and rammed in layers, and this should be done with special care if the depth of filling is considerable and the sand is not quite clean. The filling should, moreover, be done early in the working season, to allow as much time as possible for it to settle before the arch is built on it. Solid earth centerings

have been used successfully for the 55' arches of many road bridges on the Ganges Canal and they were used for some of the end spans of the Gumti bridge at Lucknow near the Medical College (5 spans of 75'). If the whole of the sand filling is not removed before the flood season sets in, it will be carried away rapidly by the first high flood provided that a central cut is made in each span to induce the current to enter the opening, but this may prove risky unless the bridge has deep foundations and is flanked by high embankments leaving the river no other course but that through the bridge.

Various periods have been laid down as proper to allow between the keying and the uncentering of arches, though it has been generally agreed that immediately after the keying of the arch the centerings should be slacked a little, so that the bricks may close in and compress the mortar. And certainly this should be done before the facing, spandrel, and outside parapet walls are built upon the arches, because a trifling change of form in the arch may occur by its settlement without impairing in its strength yet this change may crack and disfigure the external face walls but if these walls are not built until the arch has taken its final set, there will be no danger of their being afterwards deranged or disfigured. Arches have been safely uncentered immediately after keying and have changed their shape but slightly; centerings have also been left up one and two months, and even six months, and though on their removal the arches have not sunk at all, yet they have done so occasionally after the addition of the weight of the superstructure. It is clear that any change of shape in the arch must be less prejudicial to its strength while the mortar is soft than after it has set, for should any settlement *then* take place, the work must become crippled.

When, however, a large arch has been built on a solid centering, or one that cannot be properly and equally lowered, it is advisable to allow the arch to set, at least partially, before proceeding to remove such a centering, which, moreover, by its compression, has probably allowed, of some degree of settlement in the arch.

The time in which the arch has been built is an important point in this question. If the lower part of the arch, from the springing, has been made some time before the upper, it will have hardened or set, and then, if the centre is struck directly the arch is keyed, the compression of the soft part will probably cause cracks is the hardened part. But if the work has been evenly and quickly done and kept thoroughly moist, as it should be, then no doubt a gentle slight lowering of the centres is desirable to allow the arch to compress itself slightly and bring all its joints into fair bearing, but even this lowering must not be done while the mortar in the last finished joints is still so soft that it will be squeezed out.

112. **Thickness of arches**—Theoretical calculations of the thickness required for a brick or stone arch, to resist the compressive and other stresses to which it is subjected by its own weight and the load it carries, involve problems of considerable mathematical intricacy which strictly belong to the subject of Applied Mechanics, and cannot be dealt with in full detail in this Manual. They are not very satisfactory as they are based on certain assumptions, but they must be resorted to for very large arches of important structures and for arches of special design. For convenient reference, the calculations required for symmetrical arches loaded vertically, such as those of bridges usually are, have been given in Appendix B, but, in all ordinary cases of bridges of moderate span, most Engineers think that the empirical rules given in Rankine's Civil Engineering, and Molesworth's Pocket Book, which are derived from numerous existing examples of successful structures, are the best guides. These rules are given in the following paragraphs, and in the Manual on Masonry will be found a general statement of the theoretical considerations on which are based all calculations for the strength and stability of arches of different shapes under different conditions of loading.

The thickness of arches depends on the rise, on the weight supported, and on the material of which the arch is composed. French Engineers make the thickness $\frac{1}{80}$ of the span $+$ 1·1 feet, but as this rule is independent of the rise of the arch, Rankine's formula, which is based on the radius of curvature of the arch, is to be preferred. This formula makes the thickness at the crown in feet $\sqrt{0{\cdot}12\,r}$ for a single arch and $\sqrt{0{\cdot}17\,r}$ for an arch of a series, r being the radius of curve of the soffit at that point. Here again no account is taken of the material of which the arch is built. Hurst's formula is based on Rankine's formula, but gives different values to the coefficient for different materials. According to this formula, the thickness at the crown feet should be $n\sqrt{r}$, r being the radius of curvature at the crown in feet and n a constant with the values given below.

Single Arch.		*Arch of a series*	
Block stone ...	$n = 0{\cdot}3$	Block stone ...	$n = 0{\cdot}35$
Brick	$n = 0{\cdot}4$	Brick	$n = 0{\cdot}45$
Rubble	$n = 0{\cdot}45$	Rubble ...	$n = 0{\cdot}5$

The table in paragraph 116 compares the culculated and actual dimension of several successful bridges of different types.

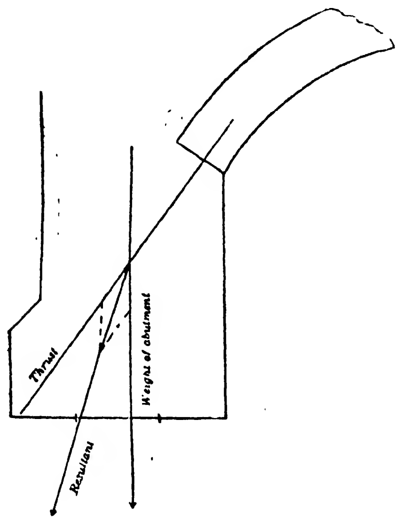
113. Arches of large span should increase in thickness from the crown to the springing. In segmental arches over 30 feet span, the thickness at the springing is usually 50 per cent. more than at the crown. Brick arches are generally divided into several sections in length, and the increase of thickness in each section is half a brick to ensure proper bond.

114. **Concrete Arches.**—Concrete arches are usually made about 10 per cent. thicker at the crown than brick arches of the same span and rise. Arches of this material act as monoliths and exert no thrust on the abutments, which need not therefore be quite so thick as those of voussoir arches. They are very useful for oblique or skewarched crossings, which are somewhat troublesome to construct in brick or stone. See the chapter on Arching in the Manual on Masonry. Concrete was used largely for the arches of all culverts up to 10 feet span on the Kangra Valley cartroad, and one of the larger bridges, of 40 feet span was also built of the same material. The concrete was composed of 1 part hydraulic stone lime, 2 sand, and 4½ broken stone. Experiments showed that concrete made of small ballast not exceeding 1″ in size was considerably stronger than that made of large material, and that the more it was consolidated by ramming, the stronger it became.

A thoroughly well constructed arch of concrete should be as strong as a similar one of brick, if composed of hard small ballast and a liberal proportion of sound mortar of cement or good hydraulic lime, but in practice the danger arises that a uniformly good quality of concrete all through the arch is difficult to ensure without the closest supervision and this it is difficult to give in large structures of this material while the inspection of brickwork is a much easier matter. The utter uselessness of inferior concrete in an arch was shown by the failure of a bridge of this material put up some years ago on the London Metropolitan Railway*, which yielded under its own weight when the centres were struck. The arch was of 75 feet span, 7½ feet rise and 3½ feet thick at the crown. It was rebuilt to the same design, but with special care as to materials and workmanship, and has proved thoroughly successful. The inference to be drawn from this case is that, without expert supervision, it is not safe to substitute concrete for brickwork under ordinary circumstances for large arches, but with close supervision we can rely upon the material whenever special conditions render its use expedient or imperative.

*See "Engineering" of 25th December, 1868.

115. Abutments of arches.—The thickness of the abutment of an arch can only be calculated theoretically if the thrust of the arch on it and the direction and point of application of the thrust at the springing, are known exactly. These are calculated as explained in paragraph 112. If the thrust on the abutment has been ascertained by calculation, the thickness of the abutment is easily found by finding the resultant of this force and the weight of the abutment acting vertically. The resultant acting on the base should be *within the middle third* of the latter if tension in mortar joints is to be avoided. The diagram in the margin requires no explanation to students who have been through their College course of Applied Mechanics.



The footings of the abutment should be splayed out to keep the maximum intensity of pressure on the foundation soil within safe limits, and the possibility of the abutment moving laterally on its base, due to the horizontal component of the resultant force acting on it, should also be carefully considered. To guard against the latter contingency, abutments of large arches on rock or very firm soil are often built with their bases inclined at such an angle as to make the bases as nearly normal to the resultant force as possible.

Another point which must not be overlooked in this connection is that the thickness of the abutment at the *springing*, where the arch rest on it, should be sufficient to prevent the layers of masonry at this level being thrust out by the horizontal component of the thrust of the arch. This force of detrusion will be resisted by the cohesive strength of the mortar bed-joins at the springing. The latter should be found by experiment in each case. For ordinary mortar it will be found to be about 2,000 lb. per square foot. With mortar of this strength the thickness at the springing should not be less than the horizontal thrust (in pounds per foot run of abutment) divided by 2,000. In abutments built up to a considerable height above the springing, or heavily loaded in some other way, the thrust would also be resisted by the friction in the joint at the springing.

In such cases, the resistance due to the superincumbent weight should also be considered, the coefficient of friction of stone or brick being assumed to be 0·7.

116. In ordinary cases of arches of small span for which detailed calculations of stresses are not considered necessary, and whose thickness has been fixed by the empirical rules given in paragraph 112, the stresses of the abutments cannot be ascertained theoretically, as the thrust is not known exactly and without it the resultant force acting on the base cannot be found. In such cases, the thickness of the abutment may be fixed by empirical formulae in the same manner as that of the arch. Two of these formulae are given below. In connection with this, the table at the end of this paragraph should be referred to; this gives the actual thickness of the abutment of numerous examples and the thickness calculated by the following French formula.

French formula—

c Thickness of arch at crown.

d span.

h Height of abutment from springing to Foundation surface.

f Versed sine or Rise of Arch.

E thickness of abutment.

H height from foundation to the top of the extrados as loaded. For road bridges the surface of the loaded extrados may be assumed to be 2 feet above the crown of the extrados of the arch, and $H = h + f + c + 2$.

1. *For Segmental Arches*—

$$E = (1 + \cdot 212\, d)\sqrt{\frac{h}{H} \times \frac{d}{f + c}}$$

2. *For semi-circular Arches*—

$$E = (2 + \cdot 162\, d)\sqrt{\frac{h + \frac{1}{4}d}{H} \times \frac{0 \cdot 87d}{\frac{1}{4}d + c}}$$

3. *For Elliptical Arches*—

$$E = (1 \cdot 4 + \cdot 154d)\sqrt{\frac{h + \cdot 54f}{H} \times \frac{0 \cdot 84d}{\cdot 46f + c}}$$

These formulae are based on the following assumptions. 1st, that in segmental arches the joint of rupture will be at the springing; 2nd, that in semi-circular arches the joint of rupture will be at an angle of 60° with the vertical, or $\frac{1}{4}d$ above the springing; this leaves the span between the points of rupture ·87d: 3rd, that in elliptical arches, the joint of rupture will form an angle of 45° with the vertical and will be at a height of ·54f above the springing; also that the span between the points of rupture will be ·84d. The numerator of the fraction having H for its denominator is the height from the foundation to the point of rupture. The numerator of the second fraction is the span between the points of rupture; the denominator is the distance from the point of rupture to the extrados of the arch.

It is understood that the thickness of abutment calculated from the formula is the *mean* thickness, which in practice may be obtained, when so desired, by the help of counterforts, the wing walls being reckoned as such when suitably placed.

Hurst's formula—

$$T = \sqrt{\tfrac{1}{5}R^2 + \tfrac{3}{5}R^{\frac{3}{2}} + \left(\frac{W}{H}\right)^2} - \frac{W}{H}$$

T = Thickness of abutment in feet.

R = Radius of arch at crown in feet.

W = Weight of 1 foot in length of half arch in cwts.

H = Height of abutment to springing in feet.

This formula gives a thickness T of abutment, without wing walls or counterforts, just sufficient to balance the thrust of the half arch, the depth at the crown being equal to $0{\cdot}4\sqrt{R}$ and the material in the arch and abutment the same. A fair margin of safety should be allowed beyond the thickness calculated by this formula.

The annexed table shows the comparison between the dimensions calculated by formula and dimensions of bridges actually built

	Bridges	Span	Versed size	Height of abutment	Thickness at crown: Real	Thickness at crown: Calculated by French Formula	Thickness at crown: Calculated by Rankine's Formulae	Total height	Thickness of abutment: Real	Thickness of abutment: Calculated by French Formulae	Radius of curvature at crown	Remarks
		d	f	h	e	e'	e''	H	E	E'	r	
	I. Segmental Arches											
1		18·1	2·3	13·1	1·8	1·5	1·3	18·9	5·9	5·8	10 45	
2		16·4	2·6	6·6	1·7	1·7	1·6	12·9	5·6	6·3	14·2	
3		25 0	2·9	14·1	2·1	1·9	2·1	20·9	11·7	11·8	28·3	
4		37·4	4·9	11·6	2 0	2·3	2·0	20·8	17·1	15·2	37·5	
5		42·7	6·1	6·6	3·0	2·5	2·6	17·2	17·1	13·9	40·4	
6	Various bridges in France and England.	46·0	6 2	20·4	3·6	2·6	3·1	31·2	19·0	19·9	45·7	
7		52·7	5·1	12·9	3·0	2·9	3·4	[illegible]	22·5	23·4	70·1	
8		53·5	10·2	20·7	2·8	2·9	2·6	[illegible]	16·0	19·0	40·1	
9		70·8	6·4	27·7	4·8	3·7	4·5	39·8	38·7	39·7	118·4	
10		82·4	11·7	17·8	4·6	3·9	3·6	35·4	31·5	30·1	78·4	
11		86·5	13·5	[illegible]	3·5	4·0	3·6	36·1	34·0	29·2	76·0	
12	Bridge at Turin ..	147·6	18·0	..	4·9	6·02	4·38	.	..	..	160·0	Single arch.
13	Grosvenor bridge over the Dee	200·0	42·0	..	4·0	7·8	4·1	..	..	..	140·0	Do.
14	Bridge of St. Maxence ..	76·7	6·4	..	4·79	3·6	4·49	..	..	..	119·0	
15	Dean Bridge	90·0	30·0	..	3 00	4·1	2·88	..	..	..	48·75	
	II. Semi-Circular Arches											
1		2·0	1·0	3·0	1·1	1·2	0·41	7·2	1·6	1·7	1·0	
2		6·6	3·3	7·9	1·1	1·3	0·75	14·5	3·9	3·4	3·3	
3		6·6	3·3	6·4	1·6	1·3	0 75	13·0	3·3	3·9	3·3	
4		9·8	4·9	11·8	1·3	1·4	1·3	20·1	4·6	4·5	4·9	
5		13·1	6·5	13·1	1·6	1·5	1·5	23·2	4·9	5·3	6·5	
6	Various bridges	16·4	8·2	9·8	1·8	1·7	1·7	21·7	5·9	6·1	8·2	
7		24·3	13·1	6·6	2·0	1·9	2·0	22·7	6.9	7·2	12·1	
8		26 9	13·4	11·8	2·5	2·0	2·1	29·3	10·5	8·3	18·4	
9		37·1	18·5	20·7	3·9	2·4	2·5	43·7	9·8	11·0	18·5	
10		49 2	24·6	6·6	3·9	2·7	2·9	35·9	12·5	12·2	24·6	
11		65·6	32·3	3·3	..	3·3	3·3	41·4	14·–	17·7	32·8	
12	Ballochmyle bridge ..	181·0	00·5	..	4·5	1·13	3·92	..	..	..	90·5	
	III. Elliptical Arches											
1		19·7	7·5	1·3	2·0	1·8	..	12·6	5·3	5·1	Unknown	
2		39·4	14·8	10·2	3·0	2·4	..	29·4	12·3	11·1		
3		[illegible]	17·1	13·6	3·7	2·8	..	35·5	14·9	14·9		
4		52·2	17·4	1·3	3·7	2·8	..	23·5	11 8	12·8		
5	Various bridges.. ..	70·0	17·5	12·0	2·3	3·4	..	34·9	16·5	21·6		
6		79·4	26·2	2·9	3·9	3·7	..	34 8	13·4	19·[illegible]		
7		80·4	27·7	6·4	4·5	3·8	..	39·9	19·	20·4		
8		115·2	34·4	3·2	6·4	4·9	..	44·5	28·8	28·9		
9		127·9	32·[illegible]	7·5	5·3	5·4	..	46·9	35·4	35·6		
10	Blackfriar's a centre arch ..	100·0	40 0	..	6·7	4·43	..	..	..	..	..	
11		55·0	12·0	8·0	3·0	2·93	2·5	24·93	13·3	17·5	37·5	
12		50·0	7·0	8·0	2·25	2·77	2·3	19·77	11·3	18·6	48·1	
13		45·0	6·0	8·0	2·5	2·6	2·5	18·6	13·6	17·2	45·2	
14	Ganges Canal bridges ..	40·0	5·4	9·0	2·0	2·43	2·6	18·83	11·0	15·75	39·7	
15		35·0	4·5	9 0	2·[illegible]	2·27	2·5	17·77	10·3	14·2	36·27	
16		30·0	4·0	9·0	2·25	2·1	2·3	17·1	[illegible]·25	12·3	30·1	
17		25·0	4·0	9·0	2·0	1·93	1·9	16·93	..	10·1	21·6	
18		2[illegible]·0	2·67	9·0	1·5	1·77	1·9	15·43	..	8·7	20·6	
19	Markunda	36·5	9·0	12·0	2·5	2·32	2·0	25·57	16·5	12·41	23·0	
20	Sohan	68·0	16·0	24·0	3·0	3·2	2·9	25·2	15·0	15·13	48·0	
21	Bridges over the Severn ..	150·0	3[illegible]·0	..	4·5	6·1	4·39	..	..	..	160·7	Single arch.
22	Ordinary bridge over double line of Railway.	30·0	7·6	..	1·88 to 2·0	2·1	1·9	..	..	..	30·0	Do.
23	London Bridge	152·0		..	5·5	6·2	5·25	..	..	..		
24	Waterloo Bridge	120·0	32·0	..	5 5	5·1	4·37	..	..	..		

117. **Spandrels**—The spandrels or spaces between the arches may be filled up in various ways, but the following are considered the best. In small bridges the masonry, having been brought up to a level with about one-quarter of the rise of the arch, is sloped up to the top of the crown, and the remaining space filled with gravel or stone-rubbish, but not with sand or clay. In large bridges, the best mode of filling up the spandrels is to built cross-walls between the arches, founded upon the solid masonry already mentioned, and increasing in length as they advance in height ; they rest upon and abut against the backs of the arches and act as struts between them. These walls are placed from two to three feet apart, and are made from 18th inches to three feet in thickness, according to their height. They are bonded together, if of considerable height, by laying long stones across from one wall to the other at intervals, or, when stone is hard to get, long flat tiles of earthenware. The outside spandrel walls, sometimes called face walls, running parallel with these, are connected with them in the same way and become a part of the general frame.

The walls are carried up nearly to the crown of the main arches, when the spaces between them are themselves arched over. Openings are made at the bottom of these walls along the top of the piers, through which any water that may fall into, or may by any means be collected in, the spandrels, is conducted to one point and issues through a pipe or spout placed for that purpose. The outside spandrel walls are usually made thicker than the interior walls, and a wall should be built along the piers and abutments crossing and binding the other walls.

When the spandrels have been brought to the proper height, they are dressed to the slope at which it is proposed to make the roadway, which is recommended to be not greater than 1 in 30.

In some large French bridges the spandrels have been filled up entirely with rubble masonry, but this throws an unnecessary weight upon the arches. In designing the spandrels it should be remembered that the tendency of a very flat arch is to fall in at the crown, rising at the haunches, and that, to counteract this tendency, a certain amount of weight is allowable and even beneficial on the haunches. On the other hand, the tendency of a high arch is to open out upwards at the crown the lower parts falling inwards and to prevent this, the haunches should be lightened as far as possible. Lack of attention to these two principles frequently leads to designs involving and excessive amount of masonry, and very needless expense is thereby incurred.

118. **Blocking course**—On top of the face walls comes the *blocking course*. This course extends the whole length of the bridge along the spandrels, arches and wing-walls. The upper part of it should be of sufficient width to allow of an inner and outer projecting ledge as well as space for the foundation of the parapet. The upper side of the outer projecting ledge or cornice should have a slope or weathering to throw off the water, and the projection should be properly throated to prevent water trickling down the face of the bridge.

119. **Parapets**—The height of the parapet wall above the roadway may be three feet in medium-sized bridges, and may be increased in larger ones to four feet. The thickness may be either a brick-and-a-half or two bricks; the former will generally be sufficient.

When there are no footpaths on the inner side at road level, there should be a projection 9 to 12 inches square, called a *wheelguard*, to keep off the cart wheels. This should be of stone if procurable, otherwise of brick-on-edge.

The blocking course, on which the parapet is built, should be from $1\frac{1}{2}$ to 2 feet wide, and one to two feet high. The inside of the parapet should be quite plain as any projecting ornament would be quickly knocked off; on the outside, at top and bottom, a neat cornice will improve the appearance of the bridge. Should a balustrade be adopted instead of the ordinary parapet wall, it should be formed of bricks moulded expressly for that purpose; hollow pottery balusters should never be used. *Perforated parapets* of various designs may be built with bricks specially moulded; these have the incidental advantage of lessening the accumulation of dust on the road.

120. **Roadway**—The roadway of a bridge should consist of a layer of brick-on-edge and, on this, a layer of good kankar or broken brick, well rammed to a depth of one foot at the centre and nine inches at the edges.

A curb of stone or brick-on-edge should be laid about four feet from the parapets for the footpaths, which may be made of paving stones or large flat tiles. The footpaths should be raised three or four inches, a sloping or saucer drain being made between the curb of the footpath and the carriage way on each side. The curb and sloping drain should be of stone when this is procurable. If footpaths are not provided, four guard stones should be fixed in the road, either at the extremities of the parapet, or at the parts where the roadway begins to narrow, so as to protect the parapets from cart wheels.

All small road bridges should be made the full breadth of the road, whatever that may be; but from motives of economy, the breadth of the roadway over large bridges may be reduced in places, where the traffic is little, to 18 feet. On all high roads, however, it should be 27 feet, and in large cities as much as 36 feet; these breadths are exclusive of the thickness of the parapets and are multiples of nine, which is the width required for the passing of a carriage conveniently and without risk of collision. Footpaths may be formed on either side, slightly overhanging the face walls, either by bricking out or by supporting them by cast-iron corbels.

When the roadway over a bridge is much higher than the adjacent country, sloping approaches may be necessary, with an inclination of 1 in 30 as a maximum, but preferably less. If the bridge has but one arch, or arches of equal height at the crown, the roadway over it should be nearly horizontal, the slopes starting from the abutments. If the arches are of different heights, the thickness of the roadway over each of them at the crown should be the same. These precautions are requisite to prevent unequal loading of the arches.

A perfectly level roadway over a bridge is less easily kept dry than one with a slight inclination both ways from the centre. By forming the road with a slight convexity in its cross-section and a slope along the length of the bridge, most of the water falling on a bridge will run along the gutters between the road and the footpath on each side; this water should be conducted beyond the end of the wing-walls, and there run into masonry drains carried in the most convenient position to low ground in the vicinity. But if this plan is not adopted and the bridge is very long, holes should be left at intervals in the sides of the bridge, under the parapets, to let the rain-water drain off the road. The bottom of each hole should be about an inch below the level of the road surface, and, if, there is a cornice, the water should issue above it, and be discharged by its drip mouldings. Drains are sometimes carried through the crowns of the arches or through the piers; obstruction is least likely to occur in the former method.

121. **Architectural features**—The architectural finish of the bridge must be left to the taste of the designer, but the following hints may be found useful.

All the ornamental features of a bridge should be bold and massive. The cornice should be full, with a good projection but with few members. A parapet looks well if pannelled, and covered with a coping curved in section. The caps of the pier heads, and end pillars of wings, might with advantage project slightly above the parapets, and the extrados lines of the arches might be indicated by projecting the arches a few inches beyond the face walls. The voussoirs of the arches might also be indicated by chiselled chambers or margins round their edges. Plasters, projecting slightly beyond the face at piers and abutments, and substantial pillars at the terminations of the wings, also produce a pleasing architectural effect.

Brick bridges are often plastered to preserve the bricks. The exposed faces of bricks are apt to crumble away in course of time, unless the face bricks are all very carefully selected, and are of the very best and hardest quality. If plaster is used, the lime should be mixed with clean sand to form the plaster and not with surkhi, which is a very perishable material when exposed, as it is usually made from bricks which have not been thoroughly well burnt. In brick bridges, a stone cornice and coping improve the appearance considerably.

122. **Loads on arched bridges**—All bridges have to support two distinct kinds of load, namely, the *dead load* consisting of their own weight and the railway or roadway which they carry, and the *moving load* consisting of the trains or vehicles and pedestrains which pass over them. The dead load is permanent and, for ordinary bridges, may be taken to be 120 lb. per *cubic foot* for the whole structure, including the arch itself, the spandrels and the roadway. The moving load is variable, temporary and may in rapid succession be evenly or unevenly distributed over the bridge, as in the case of railway trains passing quickly over it. On road bridges, the moving load is produced by a crowd of pedestrains marching in step over the bridge, or by steam road rollers and other heavy motor vehicles. The usual allowance for moving loads on *large* road bridges is 150 lb. per square foot of roadway; on *small* bridges either 150 lb. per square foot or the weight of a steam road roller, whichever is greater. The weight of the heaviest steam roller, with water and fuel, may be taken to be $16\frac{1}{4}$ tons, the steam roller being nominally one of 15 tons weight with front and rear axles about 10 feet apart. The two rear wheels, each 18″ wide; carry a greater proportion of the load than the single wide front roller, and apply a pressure of nearly $5\frac{1}{2}$ cwts. per inch of width. For railway bridges, the moving load is estimated to be as noted in Chapter VII, paragraph 217.

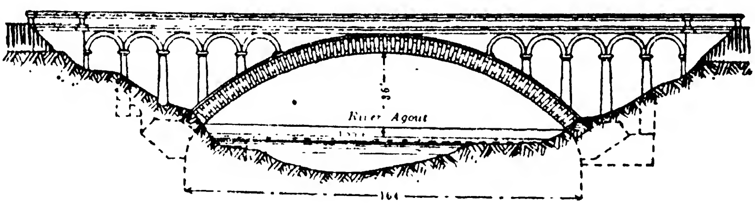
An aqueduct for a canal is the only form of bridge for which the moving load has not to be considered separately from the dead load, as the boats and barges passing over it do not impose any additional weight on the structure, since the volume of water displaced by them is equivalent to their own weight.

Wind pressure, which is another form of moving load, acting sideways, is important in high bridges of large span, but for masonry bridges of ordinary spans not exceeding 80 or 100 feet, the stresses produced by it are relatively so small that they are disregarded in actual practice.

123. **Arched bridges of large span**—Though arched masonry bridges in India are rarely built in spans exceeding 70 to 80 feet, it may be useful to record here some examples of the large bridges of this type which were erected in Europe before wrought iron had been employed in compression, or mild steel was available.

The London bridge, built of granite in 1824—1831 has a central arch with span of 152 feet, a rise of 29½ feet and a thickness at the crown of 4¾ feet. The single masonry arch of the Grosvenor bridge over the river Dee at Chester built in 1827—1832 has a span of 200 feet, a rise of 42 feet and a thickness of 4 feet at the crown and 6 feet at the springing. In France, the Antoinette Railway bridge over the Agout near Vielmar has a span of 155 feet 6 inches, a rise of 36 feet and a thickness of 5 feet at the crown and 7½ feet at the springing, see *Fig. 62.

Fig. 62. ANTOINETTE BRIDGE FRANCE.



The largest masonry arch existing is that of the Adolph bridge at Luxembourg, span 277 feet, rise 53 feet, 138 feet above river level 4¾ feet thick at crown and 7 feet at springing, 55 feet wide between parapets, and formed by two arch ribs of hard local stone, side by side the intervening space being spanned by armoured concrete. This arc was completed in 1903.

Masonry and brickwork are not, as a rule, suitable for bridges of very large span owing to the great weight of these materials in comparison with their strength, and the large heavy centerings needed for their erection.

124. **Type designs of arched bridges and culverts**—Designs of culverts and minor bridges up to 20 feet span on roads, railways and canals, are usually prepared according to type drawings issued by the

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Railway Department, and by the Public Works Department of each Province. These are readily procurable and copies will no doubt be available for reference in the College library, so it is not necessary to burden this manual with these, but to illustrate the remarks, in the foregoing paragraphs, a brief description of an arched bridge consisting of 15 spans of 63 feet over the Sohan river on the Lahore-Peshawar Road is given below. Many examples of large arched masonry bridges in India will be found in the Roorkee Professional papers of Indian Engineering.

Sohan bridge—Lahore and Peshawar Road. See plate VIII. The area drained by the Sohan at the site of the bridge is about 573 square miles, and is very compact in shape. The greatest depth of the river in floods is 15 feet, and the mean velocity about 8 or 9 feet per second. The slope of the bed is 14 feet per mile. The discharge calculated from cross-sections of the stream, is, in extreme floods, about 91,800 cubic feet per second, which is equivalent to about one-fourth of an inch over the entire catchment basin. The bed of the river exposed to view, consists of boulders; water flows all the year round and is never less than 1 foot in depth. The true bed of the river is the hard red clay of the country; over that a layer of blocks of conglomerate and over that, some 12 or more feet of boulders.

The clear waterway of the bridge is 945 running feet; the width between abutments 1,077 feet.

The design consists of 15 spans of 63 feet.

The bridge is floored throughout.

Specification—The foundations rest on the clay bottom of the river. The masonry below the flooring is of coursed rubble the faces not dressed. The floor consists of large blocks of stone well fitted, and the end stones are of the largest size cut to fit closely.

From the floor to the spring of the arches, the masonry is of the best coursed rubble in large blocks with dressed faces. The imposts are accurately cut.

The arches, spandrel walls and superstructure of parapets are of best brickwork. The cornice is of cut stone. The surface drainage of the roadway is discharged through the crown of each arch by an iron pike just clear of the wheelguard.

The entire surface of the roadway is metalled with broken stone.

The stone used is sandstone from the quarry on the Leh nallah, and none but the very best quality of stone has been put into the work.

Cost Rs. 4,28,271.

CHAPTER VI

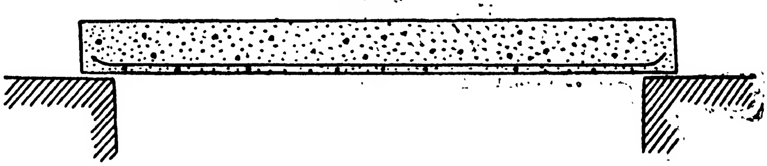
REINFORCED CONCRETE BRIDGES

125. Concrete with iron or steel reinforcement has recently taken an important place among building materials, and bridge construction is one of its best known applications. It is still not known with certainty how the two materials act together, but sufficient experience has now been gained to enable structures to be designed safely under ordinary conditions. There have been some bad failures in its use which brought discredit on it for a time, but in nearly every case where the failure has occurred investigation has shown that it was due more to imperfect design or construction than to inefficiency of the material itself.

126. A composite material of this description would tend to fail by disintegration owing to unequal expansion and contraction of the two components due to variations of temperatures, but it is a fortunate circumstance, which has made reinforced concrete a success, that the co-efficients of expansion of steel and Portland cement concrete are practically equal, being 0·0000065 for the former and 0·0000·6 for the latter.

127. The main principle on which all systems of reinforced concrete are based is, that the steel or iron embedded in the concrete is so placed within it that it will be in the best position to resist tension under stress, while the concrete takes the compression. Take for instance the case of a simple supported beam resting on two supports; in this case the metal reinforcement would be placed just above the lower edge of the beam, as in Fig. 63.

Fig. 63.



128. Before describing the practical details of construction it will perhaps be as well to consider briefly how the stresses act in this material when under transverse strain. Figs. 64 to 66 show a section of a loaded beam in which the total stresses are represented by the shaded parts above and below the neutral axis. If the beam consists of homogeneous material, like steel, the compressive and tensile stresses of which are nearly equal. Fig. 64 shows how these stresses act. The upper trian

represents the total compressive stress above the axis and the lower triangle the total tensile stress, the two being equal. In a concrete beam not reinforced, the diagram would be as in Fig. 65, as this material can stand more compression than tension; in this case the neutral axis will not be in the middle but considerably above it. If the beam is reinforced near its bottom edge by properly proportioned steel rods to assist the concrete in tension, the diagram will be as shown in Fig. 66. The resistance to rupture in this case, offered by the concrete in tension, is neglected in practice. Strictly speaking this assumption is not correct but it is generally adopted to simplify calculations, the error being on the safe side. The determination of the *exact* amount of work done by each material in resisting tension involves formulae too complicated for ordinary use.

Fig. 64.

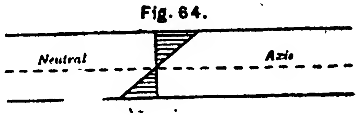


Fig. 65.

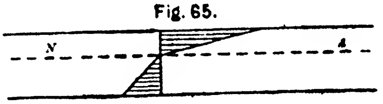
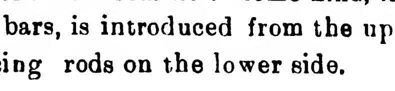


Fig 66.



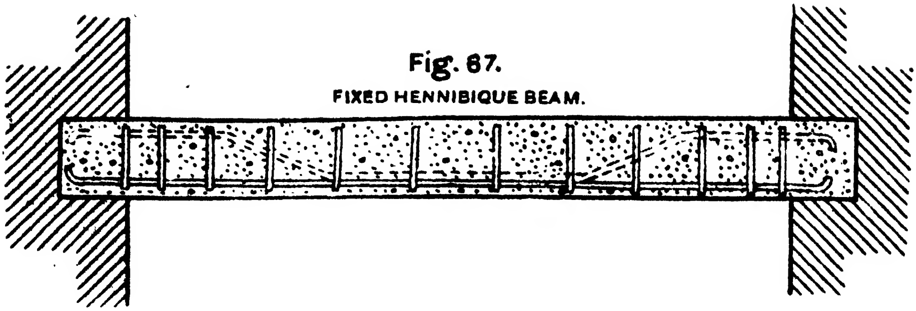
129. Besides the direct compressive and tensile stresses, the shearing stress has to be provided for in a beam under transverse strain. To meet this stress, steel reinforcement of some kind, in the shape of vertical stirrups or bent bars, is introduced from the upper side of the beam to the main reinforcing rods on the lower side.

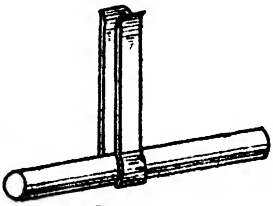
130. There are many patent systems of reinforcement, each of which has its merits, and a choice can always be made to suit the actual working conditions. The chief systems are the Hennibique. Coignet, Kahn Bar, Indented Bar, Monier, and Expanded Metal, each of which will be described briefly. Patent systems of reinforcement have certain advantages, but are too expensive for general use in India.

The **Hennibique system** is very simple and specially applicable to beams. In this system the reinforcement consists of alternate straight and bent bars placed parallel to each other longitudinally. The straight

Fig. 67.

FIXED HENNIBIQUE BEAM.





Stirrup

Fig. 68.

COIGNET REINFORCEMENT.

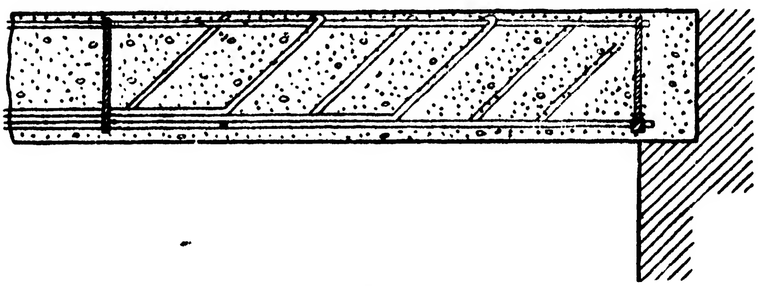


Fig. 69.

KAHIN BAR

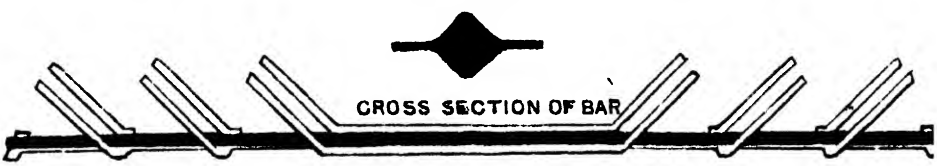
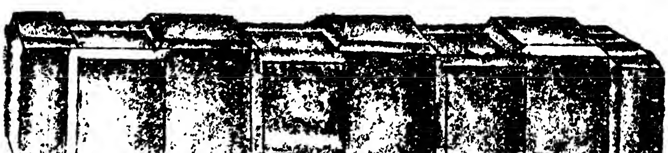


Fig. 70.

INDENTED BAR



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bars are located near the bottom of the beam along the whole length and are secured in the concrete above by flat iron verticle stirrups. The ends of the stirrup bars are turned over to fix them in the concrete. The bent bars are placed in the same horizontal plane as the straight bars in the middle of the span but towards the ends they are bent upwards to resist the diagonal tension due to shear near the supports or, in fixed beams, to meet the tension on the upper part of the beam between the points of fixation and the points of contra-flexture. See Fig. 67.

The verticle stirrups are formed of flat steel of any required thickness and width, usually $\frac{1}{16}''$ to $\frac{1}{8}''$ thick and $\frac{3}{4}''$ to $1\frac{1}{2}''$ wide.

The **Coignet system** is similar in some respects to the Hennibique but there are no verticle flat bar stirrups. The reinforcement consists of a number of round bars of small diameter grouped in fives, sevens or nines which, owing to their larger surface for the same sectional area, produce greater adhesion to the concrete than the fewer bars of larger diameter in the Hennibique system. The shear is met by the bent bars and a series of wire ties, which bind the lower tensional reinforcement to the upper part of the beam. To keep all the bars in place during concreting, and to enable the steel bars to be formed into a rigid frame reinforcing bars of small diameter are introduced into the compression flange. See Fig. 68.

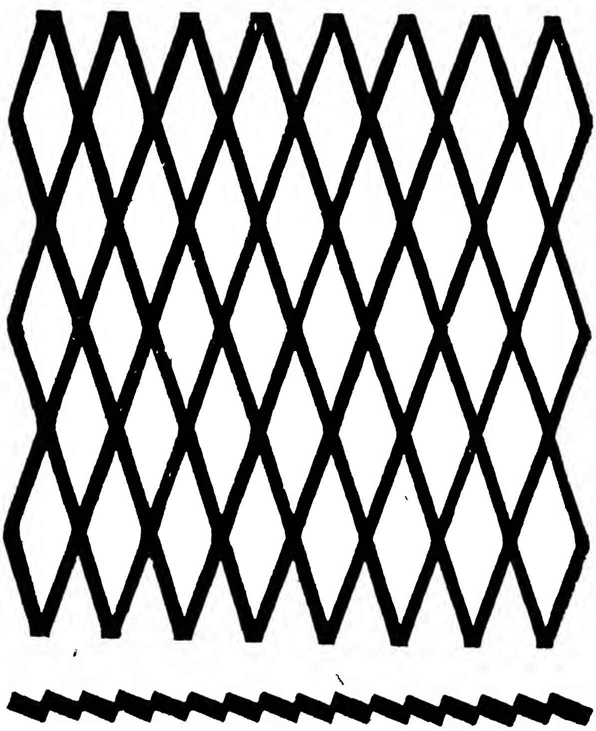
In both the Hennibique and Coignet systems the reinforcements are made of ordinary smooth steel bars and no specially rolled or forged bar are required.

The **Kahn trussed bar** is shown in Fig. 69. It has wings which are turned up at any desired angle and form a part of the bar itself. It is employed in beams in much the same way as the round bars in the Hennibique system, the only difference being that the shear members are in this case rigidly attached to the main bar. The wings not only provide resistance against diagonal tension due to shear, but being a part of the main bar, exercise a very great resistance to the bar being pulled through the concrete, and furnish a mechanical bond, in addition to producing frictional resistance, between the bars and the concrete. This system has the further advantage that at the centre, where shear is a minimum, the wings are not turned up, and the metal of which they are formed increases the section of the main reinforcing bars where the tension due to bending moment is a maximum and the greatest section of steel is required.

The **Indented Bar** system only differs from the others in the peculiar shape of the bars used. See Fig. 70. The chief object of introduction this shape of bar is to ensure a thoroughly good bond between the concrete and the steel. When used in beams the ends of some of the bars are turned up as in other systems, near the support, to resist diagonal tension due to shear.

Monier reinforcement consists of a netting of rods with rectangular meshes. The two sets of intersecting rods are called " carrying rods" and " distributing" rods, respectively. The carrying rods are the larger of the two and take most of the tensile strain, the smaller distributing rods serving to distribute the loads evenly over the carrying rods. The spacing of the rods varies with the load to be carried and may be anything from 2 to 10 inches. The two sets of rods are wired together at every third or fourth intersection. This system has been used largely for reinforcement of arches of fairly large span in Europe, principally in Germany and Austria. It is also useful for slab reinforcement. The network of rods and wires is usually embedded in a mortar composed of 1 Portland cement to 4 of sand, and not in what is generally known as concrete. *See* paragraph 144.

Fig. 71



Section of Sheet

Expanded metal is sometimes used to reinforce slabs of bridge floors on the tension side. It is made from steel plates by piercing them with parallelled rows of slits and then expanding, or stretching open, the gashes until the metal assumes the form shown in Fig. 71. Sheets of expanded metal are very convenient for use in slabs instead of Monier netting, as they can be laid in position at once, and this obviates the labour of making a network *in situ*, but their use is restricted to positions where the depth is not great as their sectional area is usually small.

131. As regards the use of bars of peculiar shape to secure a better bond between the concrete and the metal reinforcement, and thus to prevent the slipping of the latter in the former when under strain, it should by noted that many Engineers of wide experience are of the opinion that the bond with ordinary smooth bars, if these are properly placed and have their end split or hooked, is good enough for all practical purposes, and there is therefore no necessity to use special devices for improving the bond in ordinary cases; but if in any particular case there appears to be a likelihood of the bond being weakened by moisture or any other cause, it would be prudent to use one or other of the patent bars of special form which render the slipping of the rods through the concrete practically impossible, even though the adhesion is considerably reduced. In some adhesion tests recently made with "Indented" bars in cement concrete 1, 2, 4, it was found that these bars, when pulled through 8 inches of concrete, stood a tension of 1,500 lb. per square inch of surface. Ordinary round bars tested at the same time with the same concrete failed by slipping at something over 800 lb. per square inch. The allowance usually made in calculations for adhesion of concrete to metal is 100 lb. per square inch which gives a large factor of safety.

132. Some Engineer hesitate to use reinforced concrete because they fear the armouring will be destroyed by rust or chemical action, and as this material has not been in use very long it is said that it has not yet stood the test of time to justify its use. As to this, experience has shown that a coat of cement wash is one of the best preservatives of steel and it has recently been found that clean steel, which had been embedded in concrete for more than 13 years, showed no sign of rust. When taken out and examined it was found not only free from the action of rust but still blue. This shows that if iron or steel is effectually covered with Portland cement mortar or concrete, and kept free from moisture, it will not corrode for an indefinite period. To preclude all possibility of moisture penetrating through the concrete and reaching the reinforcement

it is very necessary that the concrete used for the purpose should be of a superior kind and laid with great care.

133. The fear of disintegration due to shocks or vibration has deterred some engineers from using steel concrete in railway and road bridges but experience has so far shown that there is no tendency in properly designed reinforced concrete structures to disintegrate under vibration, even when used for railway bridges of fairly large span. It is, however, advisable in the case of railway bridges to use specially long rails, without joints, over the spans, and in all bridges to provide a substantial cushion of sand or some other similar medium under the pavement or permanent way, to minimise the dynamic effects as far as possible.

Materials used and Methods of their Preparation

134. **Metal reinforcement**—Mild steel is the most suitable metal for bridge work. It should have an ultimate tensile strength of not less than 60,000 lb. per square inch. Its elastic limit or yield point should not be less than 32,500 lb. per square inch. Its elongation under breaking strain should not be less than 22 per cent. measured on a gauge length of 8 diameters. When bent cold, a bar should bend 180°, to a diameter of the thickness of the piece tested, without tearing the outside fibres. The reinforcing bars should be unwelded and in lengths as long as it is possible to get them. To make joints, the bars should be placed overlapping about 28 diameters and bound together with fine wire. The joints should occur where the stress is at its lowest.

Steel used for reinforced concrete should not be galvanised, oiled, painted, or covered with loose rust. If it is desired to protect the steel during storage it should be given a coat of cement wash.

Ordinary slight bending of small bars may be done cold, but all sharp bends and bends in bars of large diameter should be done in a forge Small bars up to 1″ diameter are more suitable than larger bars, as they have a greater perimeter for the same sectional area than a fewer number of larger bars.

135. **Cement**—Only the best cement should be used and it should be Portland cement complying with the specification adopted by the British Standards Committee; the specification is not given here as it is to be found in the Manual on Building Materials. The slow setting quality should be generally used. Every consignment received on the works should be tested. Samples for testing should be taken from one of every thirty or forty bags or barrels. On arrival at the works, the cement should be carefully stored and labelled, and, if it has been stored

for a considerable time, fresh tests should be made immediately before use, as cement varies with age; it generally improves if kept dry, but in warm, damp climates it is apt to deteriorate. The test pieces should be either 4 inch cubes or 6 inch cylinders six inches long. The compressive strength after 28 days should be at least 1,800 lb. per square inch and after 90 days 2,400 lb.

136. **Sand**—The sand should be hard, clean, coarse and sharp and the grains should preferably be of varying size. Unless the size varies somewhat there will probably be voids in the mortar and it will not be absolutely solid. Tests should be made with different samples of sand, to see which gives the best mortar, as sand is a very variable material and it is sometimes found that sand one would be disposed to reject on theoretical grounds is practically very good for the purpose for which it is required. Washing the sand does not always improve it, as the finer particles washed away are often useful in improving the compactness and solidity of the mortar. As to the size of grains it is recommended that at least 75 per cent. should pass through $\frac{1}{8}$ inch mesh and any over $\frac{1}{4}''$ diameter should be screened out.

137. The **aggregate** may be of stone or very hard broken brick (over-burnt if available). If broken brick or burnt clay is used, careful tests should be made to ascertain the strength of the concrete before applying the ordinary formulae which are usually based on concretes of hard stone aggregate. The size of aggregate used will depend to some extent on the size of the members for which it is required, but for ordinary beams and arches, the maximum allowable size is $\frac{3}{4}$ inch. The whole of the aggregate should not be of the same size but graded from $\frac{1}{4}''$ upwards to minimize voids and to obtain a thoroughly compact concrete.

138. The quality of **water** to be used in mixing depends on the temperature, the condition and the kind of materials, and the state of the weather. There should be sufficient to ensure the complete crystallization of the cement, but not so much as to convert the concrete into a slush which it is impossible to ram properly. In India the concrete should be prepared so wet that it will settle into its place, when laid, with slight tamping and will not need heavy ramming.

139. The proportions of cement, sand and aggregate generally adopted for beams and arches are 1 cement, 2 sand and 4 aggregate.

140. The concrete should in all cases be mixed in small batches and in correct proportions, and laid without delay while it is fresh. A mixing

machine should always be employed when its cost is justified by the importance and magnitude of the work. When the materials are mixed by hand, they should be turned over and thoroughly mixed on a clean platform until the colour of the cement spreads uniformly over the whole mass. The object aimed at should be to coat each grain of sand with cement, and each piece of aggregate with mortar thus produced.

141. In laying the concrete, the thickness to be punned in one operation should not exceed 3 inches, especially near the reinforcing bars, and special care should be taken to ensure close contact between the metal reinforcement and the concrete. Each section should be completed in one operation as far as possible; but when this is impracticable and new work has to be added to old, the latter should be wetted, and where it has hardened it should be chipped off, cleaned and covered with a layer of cement mortar before the fresh concrete is laid on. Work should not be carried on when the temperature is at or near freezing point, and all shaking or jarring should be avoided till the centering is struck.

Forms of bridges for which reinforced concrete is used

142. Having dealt with the different systems of reinforcement and the materials of which the reinforced concrete used for bridges is composed, we will now consider the different forms of bridges for which this material is suitable.

The simplest form is the **slab bridge**. This is limited in its application to small spans and for such unimportant structures as foot bridges and culverts, or for floors of large bridges between spandrels or arch ribs. In some cases, where headway was very limited, it has been used up to spans of 20 feet, but this is very exceptional. As a rule, single slabs are only used for spans up to 6 or 8 feet, beyond which the usual construction is a monolith consisting of a slab with stiffening ribs or girders on the under side. This ribbed construction is applied to spans up to 50 feet in length, and is particularly suitable for narrow foot bridges, or road bridges across railway lines, canals, or small streams, where headway is restricted. For longer spans than 50 feet, arch construction is invariably employed, and it is in arch bridges of large span that we find some of the best examples of the application of reinforced concrete. The arch bridge of this material occupies a position between the masonry arch and the iron or steel bridges. It possesses in a large measure the qualities of lightness characteristic of the steel bridge, while having much of the massive beauty and durability of the masonry arch.*

*Excellent models of reinforced concrete arch bridges will be found in the College Model room.

It has been chiefly used for important bridges across main streets in large towns or for carrying roadways in parks.

143. In India, the use of reinforced concrete for bridges will be limited to a great extent by the comparatively high cost in this country of the materials of which it is composed, especially the Portland cement. In most cases it will be found that masonry or plain concrete or iron structures are much less* expensive, but cases may arise occasionally in which reinforced concrete may prove to be the most suitable material to use to meet peculiar conditions, as, for instance, in bridges on important streets for which a light superstructure, is required owing to soft or doubtful foundations, but at the same time one of massive appearance and elegant design to harmonise with its surroundings. An instance of this kind is the Goomtee bridge recently built at Lucknow opposite the Medical College, of which a photograph may be seen in Plate IX, and of which a brief description is given in paragraph 170. It is possible, too, that on hill roads this material may be used occasionally with advantage for narrow foot bridges of large span over gorges, for which an iron bridge would be more expensive to construct and more difficult to maintain

Slab Bridges

144. The system of reinforcement of the plain slab is just the same in principle as that of plain rectangular beams described further on except that in bridges where heavy concentrated loads have to be provided for, distribution rods, *i.e.* rods crossing the main tension bars at right angles, are necessary to resist temperature stresses and to aid in distributing these concentrated loads over a wider area of slab than that immediately under the load. The depth of slab required to resist a certain bending moment with a certain percentage of steel, is calculated in the same way as that of a beam or girder, taking a width of 1 foot for the purposes of the calculation. The main principles on which such calculations are based have been explained in the paragraphs on beams and girders. Diagrams and tables will be found in most of the books recently published on the subject, from which the depth of the slab and proportion of reinforcement for different Bending Moments can be ascertained at once without making any calculations. March and Dunn's "Manual on Reinforced Concrete" may be mentioned as a useful book of reference for this purpose.

The Monier and Expanded Metal styles of reinforcement are very suitable for slabs, *see* paragraph 130. In the Monier system, the

* *See* Government of India Technical Paper No. 13 of May, 1909 (a Report on Tests of Concrete and Reinforced Concrete Arches, by H. F. Beale, Superintending Engineer, Public Works Department, Bombay).

mesh is generally made about 5 or 6 times the diameter of the larger rods which take the longitudinal strain. The cross rods are from $\frac{1}{2}$ to $\frac{3}{4}$ the diameter of the large ones. The following table of Monier slabs, supported, carrying $1\frac{1}{2}$ cwt. per foot superficial, in addition to their own weight, reinforced with mild steel, will be found useful.

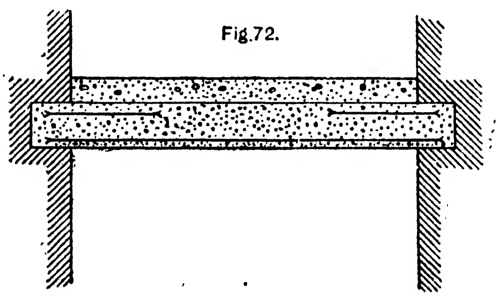
Span	Depth	Number of rods to the feet	Diameter of Longitudinal rods	Diameter of Cross rods	Remarks
Feet	Inches		Inches	Inches	
4	$2\frac{1}{2}$	4	5/16	5/32	Stone concrete. Reinforcement $\frac{1}{2}$ in above lower side.
5	3	4	11/32	6/32	
6	4	4	13/32	7/32	
7	$4\frac{1}{2}$	4	7/16	7/16	
8	5	4	7/16	7/16	
9	6	4	$\frac{1}{2}$	$\frac{1}{4}$	

If it is decided to use expanded metal reinforcement, the Engineer should obtain their latest catalogue from the Expanded Metal Company, York Mansions, York Street, Westminister, London, S. W., in which he will find tables from which he can ascertain the required thickness of slab and special reinforcing material without any calculations.

145. In taking dimensions from tables it is necessary to see what form of concrete has been assumed for calculation purposes as stone concrete is 50 per cent. stronger than that made of broken brick, and it is also important to ascertain what covering is given below the reinforcing material for adhesion. The covering should be $\frac{1}{2}$ inch at least. The working stresses assumed for the concrete and reinforcement should also be known, to enable the Engineer to check the general correctness of the tables and see how far they are suited for his purposes.

146. If slabs are fixed or continuous, the uppor side of the slab over

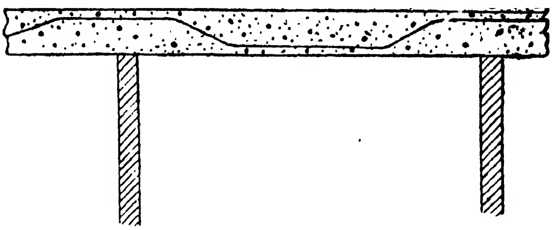
Fig.72.



the supports will be in tension for about a quarter of the span and reinforcement must be provided where tension occurs as shown in figures 72 and 73.

147. Floor slabs do not usually require special reinforcement against

. Fig. 73



shearing; the bending up of alternate bars near the ends is sufficient.

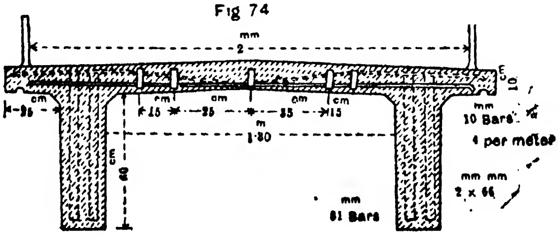
148. When a slab forms a part of a T beam and is monolithic with the rib it should first be designed with its own reinforcing bars transverse to the rib and for purpose of calculation it is necessary to consider the slab, apart from the rib, as a separate and integral part of the structure.

Girder Bridges

149. These usually consist of a flat slab stiffened by and in one piece with longitudinal ribs or girders placed 3½ to 6½ feet apart.

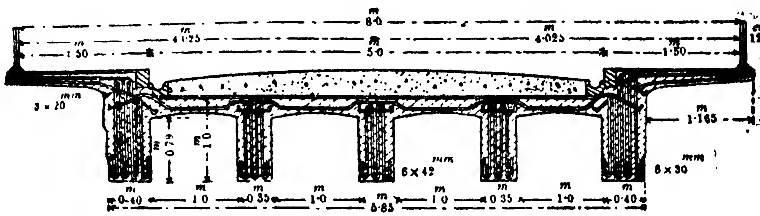
This platform may be carried on masonry abutments or on abutments of reinforced concrete. The beams are made on the same principle as slabs but as beams are of considerable depth as compared with slabs their shearing strains have to be provided for by vertical stirrups or bent bars. The systems of reinforcements for beams are numerous as explained in paragraph 130 above. The Hennibique system is the one usually adopted for spans up to 50 feet. Fig. 74 shows a cross section of a foot

Fig 74



bridge which would be suitable for a width of 6 to 8 feet, while Fig. 75 is a half section of a more important bridge, 20′ to 30′ wide, in which the roadway is carried on several ribs or girders.

Fig 75.



150. The loads on slabs and girders of roads and railway bridges will be taken to be generally those given in Chapters V and VII for arch and iron bridges, but whatever live loads be adopted it is necessary to consider the most unfavourable disposition of the load transversely as well as longitudinally and its resulting action on a floor or beam. The methods of calculation of rolling loads are the same in reinforced concrete as in other bridges but the effect of a wheel load on a thin slab of concrete required special consideration. On an ordinary rough road surface it is assumed that a heavy wheel will have a bearing of 4″ at right angles to the axle, that the load will be distributed uniformly over the upper surface of the slab through the roadway, and that the angle of dispersion of the load through the depth of the roadway will be 45° in all directions.

Fig. 76

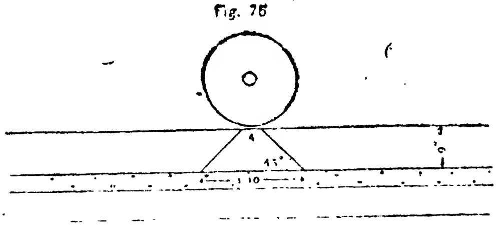
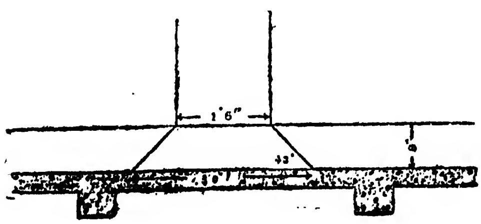


Fig. 77.



Take for example, the wheel of a traction engine 18″ wide carrying a load of 5 tons, including its own weight, on a macadam filling 9″ deep on the concrete slab. This load would be distributed over a length of the slab $= 4'' + (2 \times 9'') = 1'\text{-}10''$ and over a width of $18'' + (2 \times 9'') = 3'\text{-}0''$ so that the slab would be calculated for a strip 3″ wide with a load of 5 tons distributed uniformly over a length of 1′-10″. See Figs. 76, 77. If the bridge is a railway one, it may be assumed that the sleepers and road bed distribute each wheel load over an area measured by the width of a sleeper plus twice the thickness of road bed in one direction and by the length of a sleeper plus the same dimension in the other. The position of wheels producing the highest maximum bending moment should be considered in each case.

151. In order to increase the strength of a girder and reduce its cost, it is a common practice in reinforced concrete work to make the girder continuous or to fix its ends by extending the top reinforcements downwards into the body of the abutments.

152. If the ends of a girder are fixed it will be necessary to allow for the stresses produced by changes of temperature in addition to those due to the weight of the structure and its load. The daily range of temperature is of no consequence, as it takes time for a mass of concrete covered by filling or partially protected in some other way to be affected throughout, but the seasonal variations must be taken into account. If a range of 60° Farenheit is assumed, the change above and below the normal would be half this or 30°. The strain produced in concrete by a change of 30° would be $30 \times \cdot 000006 = \cdot 00018''$ and the resultant stress would be $\cdot 00018 \times 2{,}000{,}000$ lb. per square inch=360 lb. per square inch. The strain in the steel would be about the same and the resulting stress in the steel would be $\cdot 00018 \times 30{,}000{,}000$ lb. per square inch=5,400 lb. per square inch. In these calculations, the co-efficient of expansion of both concrete and steel has been taken to be ·000006, the modulus of elasticity of concrete to be 2,000,000 and that of steel 30,000,000. Both the compressive and tensile stresses due to contraction and elongation by changes below and above the normal should be combined separately with the same stresses due to transverse strain in order to arrive at maximum results for compression and tension in both the concrete and the steel.

153. The continuous girder is rarely employed in steel bridges but it is commonly used in reinforced concrete. The mathematical calculations of strains and stresses of continuous girders under varying working conditions are very long and involved. There are tables in text-books for continuous girder calculations where the spans are equal and the loading uniform, but in bridges the rolling loads which give the maximum strains are concentrated at certain points and not uniformly distributed, so these tables are not of much use, but graphic methods are fortunately available which are not much more complicated than those used for free

supported spans. These will be found in Church's "Mechanics of Engineering" and Du Bois' "Graphic Statics". Diagrams should be prepared in each case of the maximum bending moments produced by the dead load and the different possible arrangements of the live load. By combining these, the curves of maximum positive and maximum negative moments can be constructed, and from these it will be possible to design the reinforcement necessary to resist the tensil stress.

154. When continuous beams are of T section, the concrete in the flange or head resists the compressive strain at the centre of the span; but at the supports, while the beam is still of the same section, the moments are negative and the flange is on the tension side. The resistance of the concrete to tension cannot be taken into account, so that the section here may be considered to be rectangular of the width of the web part. There is thus insufficient concrete for the compressive resistance required at the supports, and it is necessary to make the beam deeper at this point and to introduce compression steel in the lower rods. *See Fig* 78. Reinforcement is also necessary on the upper side over the supports to take the tensile stress. The T beam, in other words, must be treated here as a doubly reinforced beam of rectangular section.

155. As already noted, deep beams or girders are liable to fail from the effects of shearing stresses, and it is necessary to introduce vertical flat iron stirrups or diagonal bent bars in the reinforcement to resist these stresses. The action of the shear reinforcement in steel concrete beams will perhaps be understood more readily if we compare it with the action of vertical and diagonal members of an open braced steel girder. The tensile stresses are taken by the vertical ties and the diagonals sloping towards the point of maximum flexure, while the compressive stresses, which are taken in the case of open girders by struts sloping towards the supports, are met in armoured concrete beams by the concrete itself. Looked at in this way, it will be obvious that diagonal ties should be rigidly attached to tension bars if they do not form part of them, as they have a tendency to slip, and in any method of shear reinforcement whether vertical, diagonal, or both, it is important to bond the ties thoroughly into the tension and compression flanges of the beam. With this object in view vertical stirrups are always hooked over the main reinforcing bars at one end and turned over at the other to give a good grip on the concrete. Where compression reinforcement is employed, the stirrups should be hooked over the compression bar as well as the tension. With diagonal reinforcement, the shear members are provided by bending up some of the main longitudinal bars towards the ends where the

reduced bending moment requires a smaller section of tension reinforcement. In such cases it is a good plan to bend the rod again horizontally for a short distance at the end to give a good key and prevent the bent rod from slipping in the concrete. The ends may also be split or hooked slightly. In fixed or continuous beams the upper horizontal portion, will, if required, form tension reinforcement for the part of the beam subjected to hogging, but this arrangement may sometimes be found inconvenient in double reinforcement when it will be sufficient to hook the upper end of the diagonal over one of the compression bars.

156. The usual depth of ┬ beams from the top of the table to the centre of the lower reinforcement is $\frac{1}{12}$ to ^{1} of the span, for, between these limits, it is found that the chances of the beam failing from shearing or bending are about even.

157. In arranging the bars of a reinforcement in a section, it is important to see that there is sufficient room between the bars or groups of bars to allow the concrete to be easily rammed into place and also to give sufficient concrete to ensure the adhesion of the steel to the concrete and thus prevent failure by horizontal shear. Bars should not be spaced less than $1\frac{1}{2}'$ diameter apart or closer than $1\frac{1}{2}'$ diameter to the sides of the beam. This applies to ordinary sections where the aggregate is broken to pass through a $\frac{3}{4}''$ ring. If the bars cannot be got into the section in one horizontal row, they may be placed in two rows and there should be vertical spacing between the rows when large bars are used. When a group of many small bars is used, vertical spacing may not be possible for practical reasons.

158. As it is difficult to understand reinforced concrete construction without a knowledge of the different stresses which such structures must be designed to meet, the author has given in Appendix C, for convenient reference, an explanation of the formulae in common use for strength of slabs and girders on which all calculations of such stresses are based. He has only taken up the simple cases of rectangular and ┬ beams under uniform distributed load.

159. The methods of calculating moments of resistance will remain the same in all cases for each particular form of section but the bending moments vary for different positions of moving loads and must be considered in detail, especially for large spans and railway bridges. The former are peculiar to reinforced concrete beams but the latter are taken out by the ordinary methods applicable to all beams under transverse strains and will not be further referred to here. If the dead load is large

as compared with the moving load, it may be permissible in some cases to treat the latter, as in the case of arch and steel bridges, by adding its equivalent to the dead load and assuming that the combined load is uniformly distributed over the whole span.

Arched Bridges

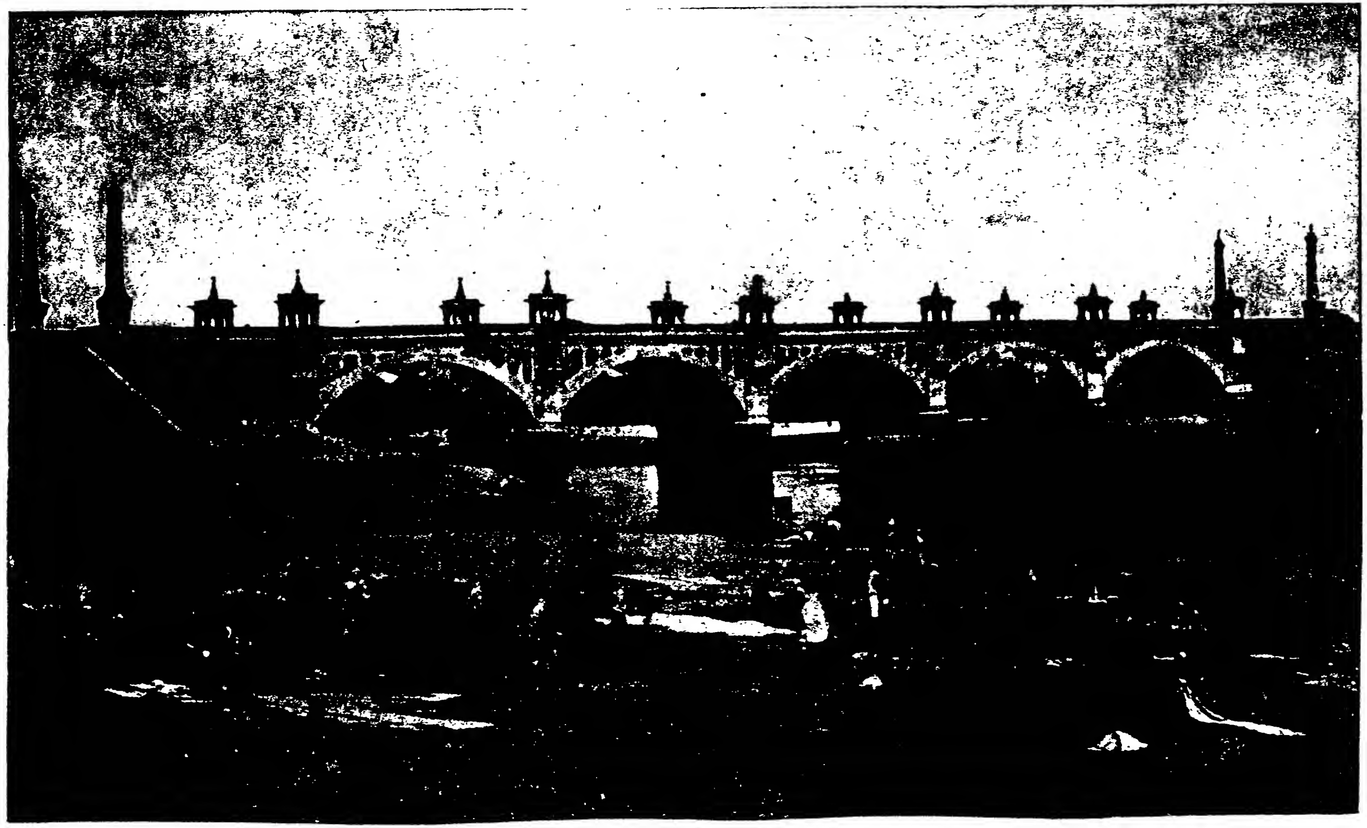
160. Masonry, brickwork and concrete arches being weak in tension have to be designed so that the line of resistance in them falls approximately within the middle third of the section. If steel reinforcement is introduced to take tensile stresses the risk of failure by tension is obviated and the thickness of the ring can be materially reduced as it is no longer necessary to confine the line of resistance within the middle third of the ring and it is possible to allow higher stresses in reinforced than in plain concrete owing to the former being a more reliable material.

161. For permanent steady loads one reinforcement only is sufficient as in Fig. 79 and this in fixed arches should be carried well into the abutment. But, as, under unequal loading, arches may fail through tension at the extrados near the springing, a double reinforcement should be provided to meet it. The upper reinforcement is sometimes provided near the springing only as in Fig. 80 but as the actual point of rupture may vary, it is desirable to carry the armature over the whole arch as in Figs. 81, 82.

162. In addition, stirrups are introduced in large arches to tie the reinforcements together and resist any tendency to shear. By distributing the stresses, these stirrups moreover help the main reinforcing members to resist the deformation to which as curved bars, they are liable under longitudinal strain. Without such support there is obviously a tendency in these bars to straighten out under tension and to buckle under compression Three examples are given below showing how the shearing reinforcement may be provided ; Fig. 83 shows the Melan system, Fig. 84 the Hennibique, and Fig. 85 the Kahn bar.

163. The abutments of reinforced concrete arches must be designed strong enough to take the thrust at the springing and in Europe and America they are often designed suitably in reinforced concrete. They may however be of masonry or brickwork and in India this form of construction will probably be found to be more economical in most cases.

Plate IX



Bridge over the Goomtee River at Lucknow

164. In large arches the weight of the superstructure is considerably reduced by omitting the thick spandrel walls or filling which usually support the roadway in ordinary masonry bridges and substituting either thin transverse walls or light reinforced columns resting directly on the arch ring. The roadway in such cases is carried on a series of minor arches spanning the distance between the walls or columns or on a slab floor resting on them. This arrangement lends itself well to artistic treatment and produces a pleasing architectural effect. *See* Plate IX.

165. Owing to the lightness of a reinforced concrete arch and its superstructure, much larger spans are possible in this material than in masonry brickwork or concrete. With increasing experience, the spans of armoured concrete arches are gradually becoming greater. The largest yet completed is that of the Grafton Arch in Auckland, 350 feet.

166. The disposition of the reinforcement is more varied in arches than in other works of this material. Several systems have already been referred to above. The three systems most commonly employed in the large spans erected so far in Europe and America are the Monier, the Hennibique and the Melan all of which have been already explained.

167. There are two other systems not yet referred to which are getting common and deserve special mention—the Considere and the Ribbed Ring.

168. **Considere system**—As an arch has to be treated chiefly as a pillar under direct compression in considering the stresses to which it is subject, it is obvious that the means adopted for strengthening and stiffening pillars are also applicable to arch rings. The strength of a pillar is increased considerably by longitudinal and lateral reinforcement and experience has shown that lateral reinforcement is the more effective of the two weight for weight. The most efficient form of lateral reinforcement at present known is the spiral or helix which winds round the section in series without actually encircling it. This system of reinforcement is called the Considere after the eminent French Engineer who first used it. *Figs.* 86, 87 show how it is applied to arch rings.

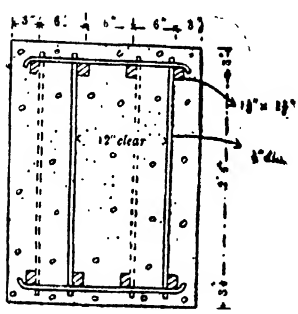
169. **Ribbed rings**—Many important bridges of large span have recently been built by constructing the arch in several ribs connected together and stiffened laterally by narrow reinforced concrete stiffeners between the ribs where the columns or thin transverse walls come. This method of construction is very economical and in some cases is said to have saved 25 to 30 per cent. of the cost of the ring. The consequent reduction in weight is a further advantage and tends to reduce the cost

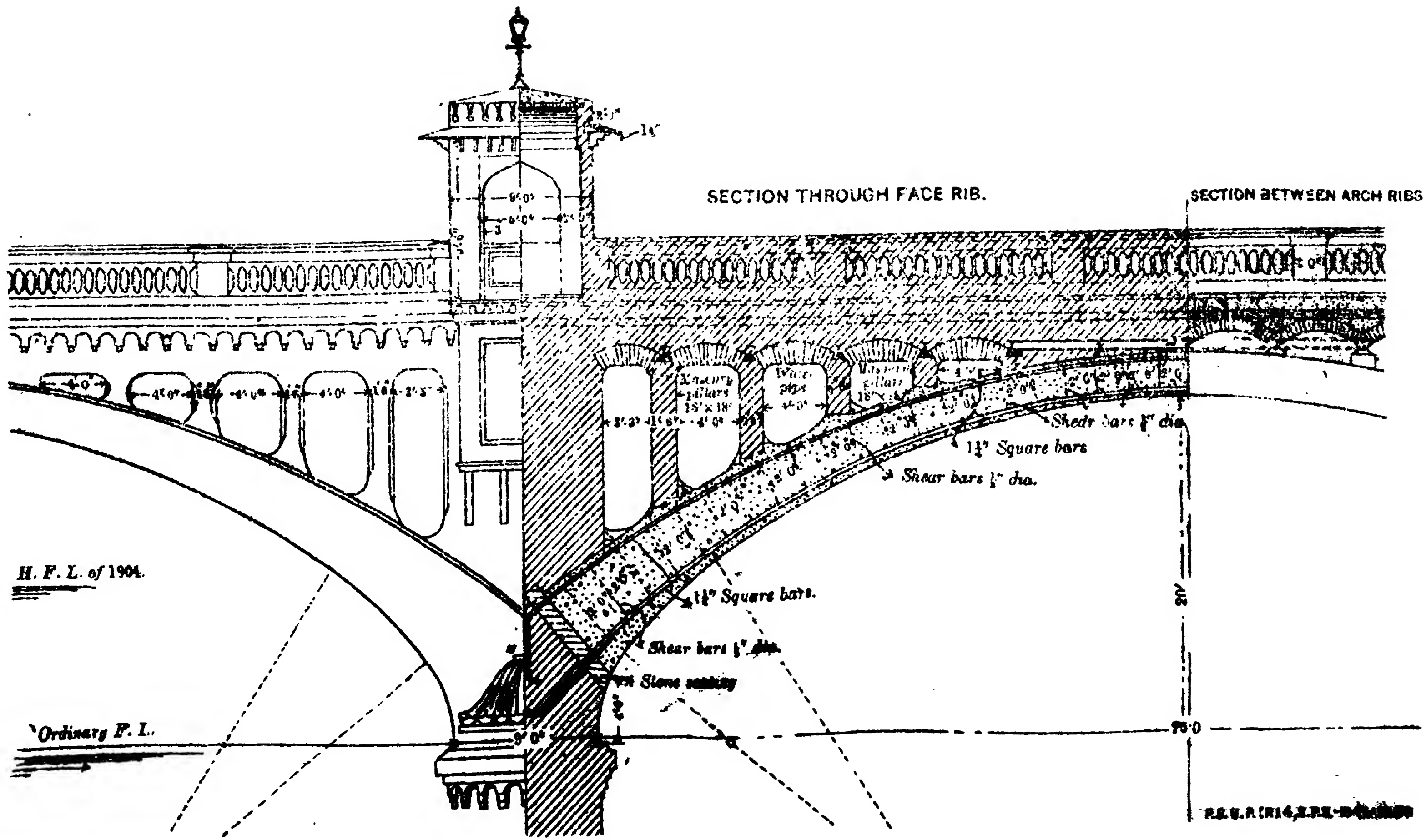
of the abutments and load on foundations By giving the ribs a radial depth somewhat greater than would be used for a ring of the full width of the bridge, a more economical distribution of material results for strength and stiffness. The Goomtee bridge, of which a photograph may be seen in Plate IX, is built in this way.

170. The span of this bridge is 75 feet and the rise of the arches 20 feet. There are seven ribs 4 feet apart and 2 feet wide with a depth of 2′ 6″ at the crown and 5′ 0″ at the springing. *See* Fig. 88. The roadway is 34 feet wide between parapets.

There are four cross-stiffeners between the ribs opposite each line of pillars. These are the full depth of the rib and nine inches thick, 5′ 6″ apart centre to centre. The pillars are 18″ square of reinforced brick-work; their vertical reinforcing bars (½″) are carried down through the depth of the ribs. The pillars are spanned across the bridge by 7″ steel joists and 9″ jack arches are turned on the joists. The reinforcements consist of four 1¼″ square mild steel bars both on the extrados and the intrados spaced six inches apart centre to centre and about 2″ from the upper and lower surface. The shear reinforcement consists of four radial ½″ rods connected to the corresponding upper and lower main reinforcing rods by means of 16 gauge soft iron wire and also to cross bars top and bottom passing across the latter and hooked round at the ends as shown in Fig. 89 to keep them in position. The shear bars

Fig 89
CROSS SECTION
OF
RIB
AT CROWN SHOWING
SHEAR BARS.





SECTION THROUGH FACE RIB.
SECTION BETWEEN ARCH RIBS
Shear bars ⅝" dia.
1¼" Square bars
Shear bars ½" dia.
1¼" Square bars.
Shear bars ½" dia.
Stone sealing
H. F. L. of 1904.
Ordinary F. L.
Masonry pillars 18" x 18"
Water pipe

are placed 2′ apart along the length of the arch. The ends of the main reinforcing rods were secured to the masonry of the skewbacks, abutments and piers as shown in Figs. 90, 91.

Fig 90

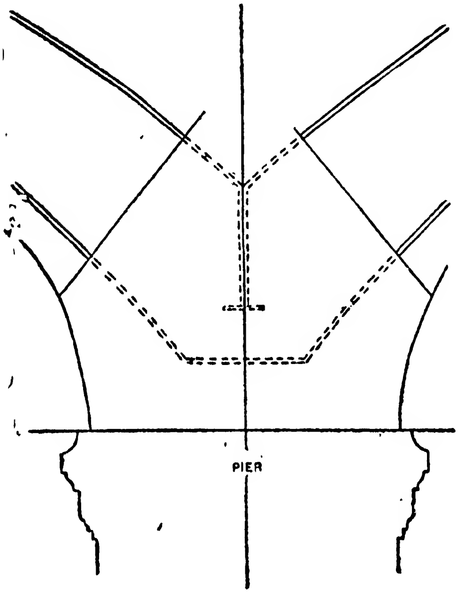
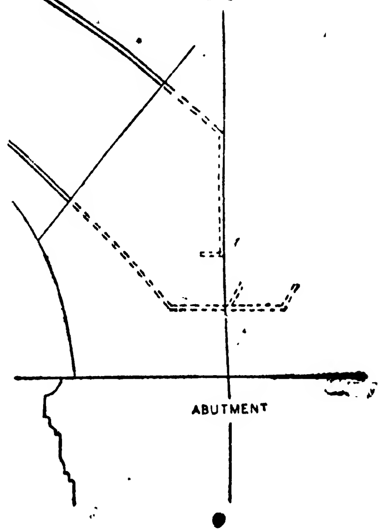


Fig 91.



171. Several narrow reinforced concrete bridges for local roads have recently been built in the Terai of even greater span than the Goomtee bridge. The Baror and Baur bridges have 80′ spans, the Dabka bridge 120′ and the Baigul bridge 200 feet. These will be found fully described in paper no. 16, read in 1913 at the Simla Engineers' Conference by Mr. W. L. Stámpe, Executive Engineer, who built them. A few leading details are given below for convenient reference:

80′ Spans—Rise 16′. Width of ribs 15″. Depth at crown 2′ 0″, at springing 3′ 5″, clear distance apart 4′ 0″. Number of ribs 3. Width of roadway 10′. Reinforcement three bars 1¼″ × 1¼″ square mild steel, top and bottom. Vertical pillars 18″ × 15″ re inforced by four ½″ bars carried through ribs and covering girders, transverse stiffening ribs 6″ thick, one at abutments, one at each pillar near abutments 4′ 6″ apart, and one at crown, reinforced by two ½″ bars below and two above. Roadway girders 15″ wide, 9″ deep, one over each row of pillars, reinforced by *three ½″ bars below and three above, monolithic with the floor slab which is 4″ thick reinforced by ½″ *bars 9″ apart laid transversely on the lower side with 3 feet continuity bars *½″ diameter 9″ apart on the upper side over supports. Roadway of stone metal 9″ thick. Live load on arch ring 150 lb. per square foot, con-

*See note on cross-section as to these dimensions.

centrated local load on road platform 250 lb. per square foot. Extreme temperature range 60° (80° above normal and 30° below). Arches fixed at abutments:

120′ Spans—Rise 24′. Width of ribs 15″. Depth at crown 2′ 6″ at springing 4′ 6″. Reinforcement three mild steel bars top and bottom at the crown and four similar bars at the springing for a length of 20 feet. All other details exactly the same as those for the 80′ span.

The reasons given for constructing light reinforced concrete bridges in this district in place of masonry or steel are as follows:

"Owing to the unstable nature of the subsoil, many of the old masonry bridges have cracked owing to settlement of foundations and the steel bridges have given much trouble both on account of the difficulty of erection and the necessity for frequent painting to preserve the steelwork."

As a large number of bridges had to be renewed in the district at the same time, great economy was effected by standardising the designs and making the same set of centering and falsework do for many bridges. The 80′ spans were erected at a cost of 100 rupees per foot run, including new foundations, and the 120′ spans at 110 rupees per foot run.

Cross and longitudinal sections are given in Figs. 92, 93 to show the arch construction of an 80-feet span. The points to be noticed in the designs are: (1) the use of a few bars of somewhat large section instead of a larger number of small rods. Small bars up to 1″ diameter are preferred by many Engineers for reinforced concrete work, as they distribute the stress more uniformly through the concrete and there is a larger surface of adhesion per square inch of section; but, on the other hand, the labour of placing and fixing a large number of rods increases with the number used and it was probably this consideration that influenced the Engineer in deciding to use the large sections; (2) the absence of stirrups or radial steel reinforcements. The shearing stress being small, the concrete has been relied on to take it but with reference to this *see* paragraph 162. Here again the desire to avoid complications in the work as far as possible has probably induced the Engineer to omit all radial reinforcements; (3) the use of unhinged arches in a position where the nature of the sub-soil is said to be unstable, *see* paragraph 172. The foundations have no doubt been secured and the Engineer has satisfied himself that there is no likelihood of any future settlement.

172. Hinged Arches—It is a common practice on the Continent to use arches with two or three hinges or articulations. These arches have the drawback of being less rigid and therefore more liable to vibration from rolling loads, but, on the other hand, they have the

Fig. 92.

FRONT ELEVATION. HALF SECTION.

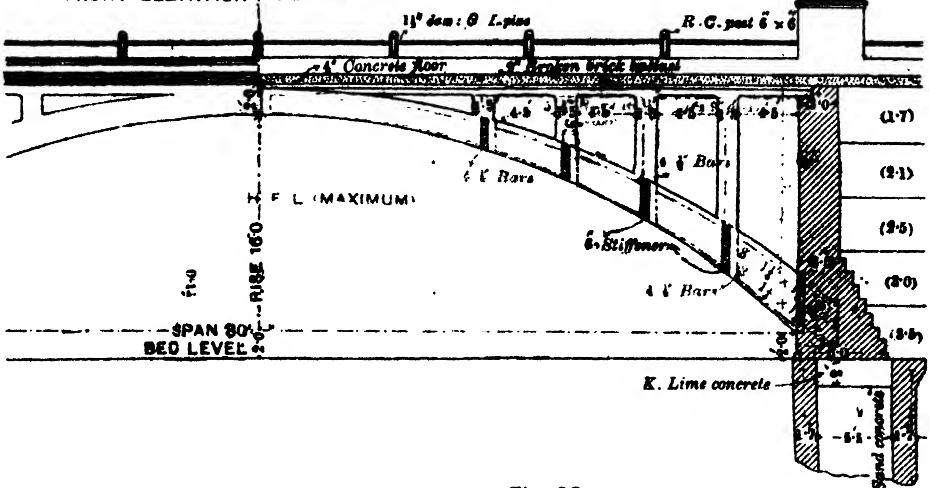
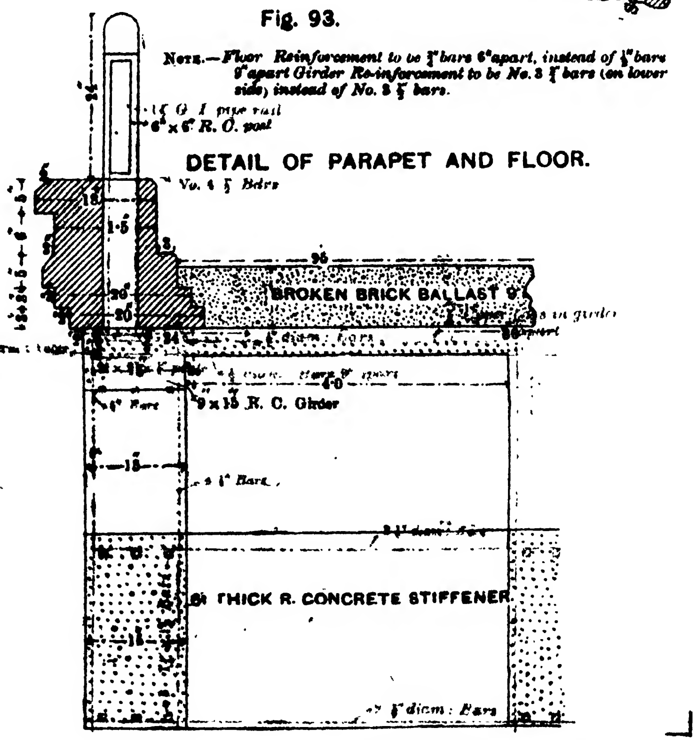


Fig. 93.

NOTE.—*Floor Reinforcement to be* ⅜" *bars* 6" *apart, instead of* ½" *bars* 9" *apart Girder Re-inforcement to be No.* 3 ⅞" *bars (on lower side) instead of No.* 3 ⅝" *bars.*

DETAIL OF PARAPET AND FLOOR.



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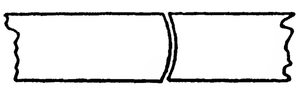
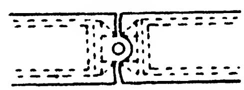
advantage of the greater accuracy with which their stresses can be determined and of being safer when there is any possibility of settlement of foundations. With arches having no hinges, any slight yielding of the abutments has a very serious effect on the stresses. With three hinges the stresses in an arch can be determined without recourse to the very complicated elastic theory which involves much labour in calculations. In spite of the advantages of the hinged arch mentioned above, many English and American Engineers prefer to spend a little more on the foundations to make them secure and avoid the complication of hinges.

The hinges used in bridges may be of granite or some other very hard stone, or of steel. When of stone, one of the two stones forming the hinge is made circular and convex and the other circular and concave with a slightly larger radius to receive it. When of steel, they consist of cast steel bed-plates and hinge pins.

Stone hinge

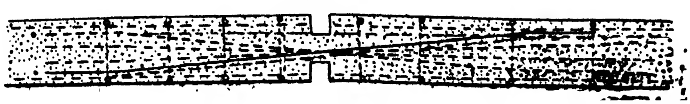


Steel hinge



173. A method of forming hinge joints peculiar to reinforced concrete arches is that of semi-articulations. By this method, the sections of the arch are so reduced at the hinge points that the line of stress is compelled to pass through very small and well defined sections. The line of pressure is thus exactly located. In a true hinge joint there may be considerable friction preventing rotation, but in these joints the resistance to rotation is assumed to be so small as to be negligible. The section of concrete being greatly reduced at the hinge points, additional rods have to be introduced into the concrete, and grouped together so as to pass through this small section. At the section, there is great resistance to compression but very little to bending. *Fig.* 94 is a sketch of the semi-articulation used in a 90-feet arch over the canal St. Martin in Paris, recently built on pre-existing abutments, the absolute rigidity of which against settlement was doubtful. The arch was tested after erection and the results of the test are recorded in the "Annales des Ponts et Chaussees, 1907, Vol. II, page 180.

Fig. 94.



174. **Calculations of Arch Rings**—Experiments which were made under the direction of the Austrian Society of Architects and Engineers on full-sized arches, have shown conclusively that the theory of the elastic Arch, as it is called, may be applied with confidence to reinforced concrete arches. In these experiments it was found that the calculated elastic deflections corresponded with the actual deflection. This theory has been much simplified of late years, and the labour involved in its use has been greatly reduced. The analysis of the elastic arch without hinges has been demonstrated very clearly by Professors Maurer and Turneaure in "Principles of Reinforced Concrete" and will also be found fully explained in "Reinforced Concrete" by Marsh and Dunn, or in a book of the same title by Buel and Hill or in "Extracts on Reinforced Concrete Design" by Taylor and Thomson. Useful examples of calculations in full detail will be found in the printed estimate of the Goomtee bridge by Mr. A. C. Verrieres, C. I. E., Personal Assistant to the Chief Engineer, United Provinces, and in paper no. 16 read at the Simla Engineers' Conference in 1913, by Mr. W. L. Stampe, Executive Engineer.

175. The elastic theory affords an exact method of calculating the stresses in an arch of any form or style, and of constant or variable cross-sections, that is fixed at the skewbacks and continuous at the crown. It may also be applied to arches hinged at the ends only, for which the solution is very simple if the hinge friction is neglected. No attempt will be made in this Manual to explain this theory mathematically, as the student will no doubt take this up in his course of applied mechanics but it may be briefly noted here that the method of calculation based on this theory consists in finding the true equilibrium polygon for any given arch and system of loading, and its true position in the arch ring. The conditions to be fulfilled to obtain these results are (1) that the tangents to the neutral line of the ring at the springing are fixed in direction, (2) there is no change in the span of the arch, and (3) that the vertical deflection of one springing line with respect to the other is nil. Equations are deduced from the above conditions which give the thrust, shear, and moment due to dead and live loads, as also the thrust and moment due to temperature variations, and to rib shortening under pressure. When these stresses have been obtained and brought together, the arch is calculated as a column loaded eccentrically, and subject to compressive stress due to direct compression and transverse strain.

176. **Though elaborate calculations of stresses in large arches subject to heavy moving loads of varying intensities would be out of place in this**

Manual, the author has given in Appendix C, some simple calculations which may be applied to arches, the form of which is parabolic or a segment with a small rise and approximately parabolic, under a load uniformly distributed over the whole span, or over half the span on one side. These calculations are approximate and should only be used for small spans, or for roughly fixing the dimensions of a trial arch before making detailed calculations to see if it is of sufficient strength.

Centerings and Falsework

177. Successful arch construction and its cost depend as much on the design of the centering and falsework as upon the design and construction of the abutments, piers and arch ring.

The supports for the centres should be as unyielding as it is possible to make them. If piles are used, they should be as well driven as permanent foundation piles, and sills should only be used when the ground on which they rest is absolutely firm and unyielding. The construction of centers will not be further referred to here as they have already been dealt with in Chapter V on Arched Bridges of Masonry and in the Carpentry Manual.

The mould frame or "falsework", as it is usually called is reinforced concrete work, should be (1) rigid, (2) simple in construction and easily erected and removed, (3) so arranged that the surfaces will not deform the concrete by the expansion of the moulds when wet, (4) so designed that the frames can be used over and over again in different parts of the work, (5) so prepared that it will come away easily without damaging the concrete when it is removed.

For moulds or falsework, the timber selected should not be too dry, as it is likely to swell irregularly when it becomes wet by contact with the fresh concrete, but on no account should green timber be used. All faces against which the concrete will be placed should be planed smooth, and be free from knot holes and other imperfections.

To ensure rigidity in the falsework, the timber used should be of sufficient scantling, and its support and bracings should be carefully designed. It is better to use timber too thick than too thin for this purpose. The boards for floor slabs should be, as a rule, 1½ inch thick, sides of beams 2 inches, and sills 2 to 3 inches.

The timber should be so arranged that it can be easily dismantled and re-erected, and with this end in view bolts and clamps should be used rather than nails or spikes, and mortices, tongues and wedges should be freely used at joints. Deformation of the concrete surfaces may be prevented in many ways.

(*a*) Metal moulds or moulds lined with metal may be used.

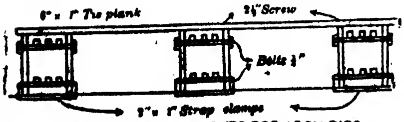
(*b*) If timber moulds are used, the joints may be tongued and grooved, or the joints may be left slightly open and the mould lined internally with jute, canvas, or oiled paper.

(*c*) The boards may have splayed edges so that the splayed edge of one board can slide over the adjacent plank, if expansion occurs, without causing any appreciable deformation of the concrete face.

To prevent the boards adhering to the concrete and damaging it when removed, the timber surfaces which will be in contact with the concrete should be coated with soft soap or mineral oil. Fatty oils act on the concrete and should not be used for this purpose. The use of oiled paper, or paper and soft soap, as an inside lining, will prevent adherance and also cover gaping joints in the boards, if there are any.

178. The side falsework is generally removed 4 to 5 days after the concrete has been laid. The support under the arches is removed after 28 days as a rule, but for very large arches it may be left up another week or two.

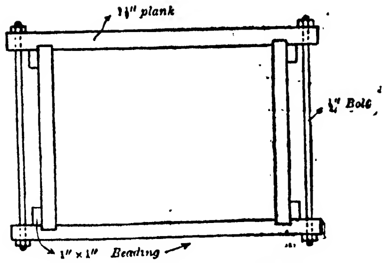
179. The design of falsework will of course vary greatly to suit the conditions in each case, and it is not possible to lay down hard and fast rules as to design, but with the hints given above, the young Engineer should find no difficulty in getting out a suitable and economical design for his moulds. A visit to a bridge in course of construction will teach more than can be learnt from volumes of instructions, and if such a visit is possible it should certainly be undertaken if a sound practical knowledge of this part of the work is desired. The student who has mastered the main principles laid down above will soon grasp the reasons for the many designs and devices employed in this class of work. An extract from Mr. Stampe's paper referred to in para. 174 is given below, describing the falsework used by him for the bridges he has recently built in the Terai; this should be useful to those who have not had on opportunity of visiting a reinforced concrete bridge under construction.



SKETCH SHOWING BOXES FOR ARCH RIBS

Side shuttering of 1½ inch salwood planks stiffened at 4′ intervals by 6″×2″ battens has been found to be the most efficient form of shuttering, but is expensive. If several bridges of the same span are to be built as was recently the case in the Terai, it will be found economical to employ salwood shuttering. For individual bridges sambhal planks stiffened alternately by salwood battens and 2″×2″ angle irons have been ound economical.

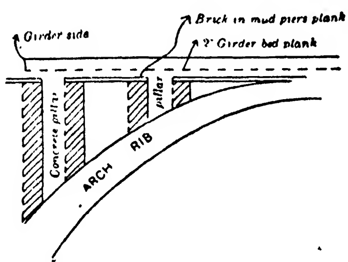
To stiffen the boxings and preserve the parallel alignment of the ribs 6′×1′ wooden stays are fixed at 3 feet intervals transversely over the ribs, and screwed to the top of the side shuttering. Steel cross bolts ¾ inch diameter serve the double purpose of supporting the reinforcement bars at their correct radical depths and of preventing outward displacement of the shuttering by the pressure of the concrete.



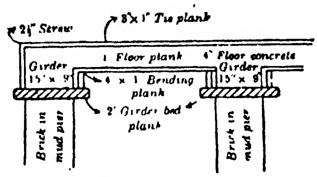
SKETCH PLAN OF COLUMN BOXES

It has been found perfectly feasible to wire the reinforcements loosely to the cross-bolts in order preserve their alignment during concreting, and to knock out the bolt and grout up the cavity when the shuttering is dismantled.

A double series of 2″×½″ iron clamps, above and below the side shuttering, is also employed to prevent spreading of the extremities of the planks.



SKETCH SHOWING GIRDER FALSE WORK ELEVATION



SKETCH SECTION OF GIRDER AND FLOOR FALSE WORK

On the completion of the arch ribs the column concrete is poured in a semi-liquid state into the vertical column boxes. These are usually made up of 1 inch or 1½ inch sal planks bolted together as shown in the marginal sketch. The boxes rest on the ribs at their lower ends and are supported by a series of cross-stays.

The falsework for the road girders and platform is erected after the completion of the concrete columns.

The 2′ sal bed planks for the girders are usually carried by brick in mud pillars (1·5′ × 0·8′ in section) built up from the arch rib on either side of the columns as shown in the sketch.

The platform concrete is supported by 1′ or 1½′ transverse planks carried by a sal beading fixed to the side planks of the girders.

CHAPTER VII

IRON AND STEEL BRIDGES

180. The strength and durability of iron, whether in the form of cast iron, wrought iron or steel, render it extremely valuable for the superstructure of bridges, and the improvements recently made in the processes of its manufacture have led to the gradual supersession of timber and masonry for this purpose in all important bridges of large span. The earlier stages of the development of iron bridges were marked by improvements in the method of conversion of cast iron into wrought iron, a more reliable material. Sections of wrought iron were produced which were well suited for the construction of iron girders, and more recently, the experience acquired in the manufacture of *mild steel* by the Bessemer and Siemens-Martin processes has accumulated so fast that a material of even greater strength, ductility and elasticity is now produced at a cost almost equal to that of wrought iron.

Materials of Construction

181. It is not necessary in this Manual to describe the qualities and characteristics of cast iron, wrought iron and steel as these are fully dealt with in the Manual on Building Materials, but it may be useful to repeat here the average ultimate strength and working stresses of the three materials under the strains to which they are usually subjected in the framed structures of bridges.

The average ultimate tensile strength of cast iron is 8 tons per square inch, of wrought iron 24 tons, and of mild steel 30 tons. The ultimate resistance to compression of cast iron is 40 tons per square inch, of wrought iron 17 tons per square inch and of mild steel the same as that under tension, *viz.* 30 tons.

The working stresses of cast iron when subjected to a dead load are generally taken to be $\frac{1}{4}$ of the ultimate strength, or when subjected to shocks or impact, $\frac{1}{10}$, the breaking load. The working stresses of wrought iron and steel are usually taken as $\frac{1}{3}$ breaking load for permanent dead loads not liable to variation, $\frac{1}{4}$ to $\frac{1}{7}$ for ordinary live loads gradually applied and $\frac{1}{8}$ to $\frac{1}{10}$ for live loads rapidly applied.

The *shearing working stress* of rivets, bars and plates of rivetted joints in wrought iron and steel is generally assumed to be 6 tons per square inch and the safe bearing resistance 8 tons per square inch.

182. **Testing materials**—For all important work, the Engineer should satisfy himself that he is getting material of good quality from the manufacturers by making certain recognised tests which are intended to show that the iron or steel supplied has the required degree of strength and ductility. The following are the tests prescribed by the Director-General of Stores for India for the material of bridges for State Railways made in England. In ordering wrought iron or steel, it is generally specified that a certain number of samples of the materials must be provided by the makers during manufacture to be tested by the Engineer or his Inspector.

	Tensional stresses per square inch	Percentage of contraction of fractured area	Percentage of elongation in a length of 10 inches
Wrought iron plates and bars	**Tons**		
Square and round bars and flat bars under 6″ width.	24	20	15
Angle and Tee bars and flat bars over 6″ width	22	15	12
Plates—with the grain	21	10	8
Do. across the grain	18	5	4
Steel plates and bars			
Plates, with or across the grain and angle or Tee bars—			
not less than ..	27	..	20
not more than ..	31		
Steel rods for bolts and rivets—			
not less than ..	25	..	25
not more than ..	28		

Rivet iron is tested as follows:

(*a*) Bending double upon itself when cold.

(*b*) Bending double upon itself when red hot.

(*c*) Shank being nicked whilst cold and bent double, showing the fibre of the iron to be of good quality.

(*d*) Flattening down the rivet head whilst red hot until its diameter is 2½ times that of the shank without showing any sign of cracking at the edges.

183. Cast iron is now but seldom used for structural purposes as its tensile strength is very small compared with wrought iron and mild

steel and it is not a reliable metal being brittle and liable to give way without warning. It is only used occasionally in parts of structures subject to direct compressive stress without vibration. Where its strength is of particular importance, test bars should be moulded from the same metal as that employed in the casting and they should be tested by loading them as beams. Test bars $2'' \times 1''$ section, placed on supports 3 feet apart with the $2''$ side vertical, should sustain a central test load of 30 cwt. with a deflection of not less than 0·3 inch previous to fracture.

184. **Section and stock sizes of material**—Cast iron can be moulded to any shape or size. Wrought iron and steel are rolled in a great variety of sections to suit the different patterns of girders employed in ordinary engineering practice. For convenience of classification, the following nomenclature has been adopted for the sections in common use:

"Plate", for thicknesses of $\frac{3}{16}''$ and upwards when width is above 12 inches.

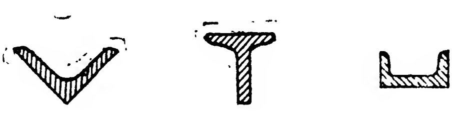
"Flat", for thicknesses of $\frac{3}{16}''$ and upwards when width is not above 12 inches.

"Sheet", for any thickness less than $\frac{3}{16}''$ above $6''$ width.

"Hoop", for any thickness less than $\frac{3}{16}''$ not above $6''$ width.

"Bar", may be either square or round in section. Ordinary dimensions are from ½ inch to 3 inches diameter or sides, increasing by $\frac{1}{16}$ of an inch each size. If under ½ inch diameter, round bars are called rods. If under $\frac{3}{16}''$ diameter wire.

"Angle", Tee and Channel. Those are shown below:



These sections are made in a great variety of dimensions. Iron merchants generally publish lists showing those they keep in stock.

"Rolled joist or H. iron."

This section is rolled in depths of $8''$ to $18''$. An endless variety of sections is kept by different makers who generally publish full size sections of their joists showing the weight per foot run of each joist the stock lengths and the distributed load each section will support.

Besides the above-mentioned, there are a great many other forms which are rolled for special purposes such as bulb bars, single headed rails, double headed rails, bridge rails, etc., which are usually shown by makers in their catalogues and price lists.

The ordinary commercial length of angles, tees, channels, joists and bars is 20 feet but they are obtainable in lengths up to 40 feet. They can be rolled to a maximum length of 60 feet in steel but only to special order. The limiting dimensions of plates are as follows:

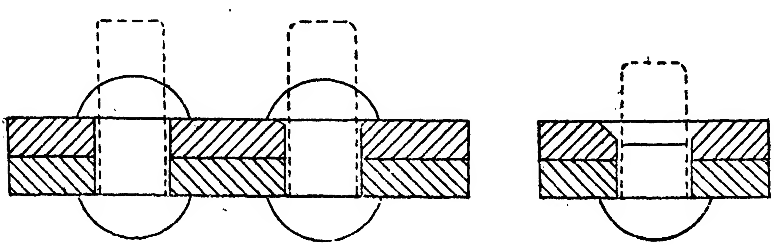
Thickness, $1\frac{1}{4}$ inch; length in steel 40 feet, in wrought iron 14 feet; width in steel 10 feet, in wrought iron 8 feet.

In bridge work, in ordinary practice, it is not advisable to use plates less than $\frac{1}{4}$ inch thick in any part of the structure, or, for diagonals, round iron less than $\frac{3}{4}$ inch; flat bars less than $\frac{1}{2}$ inch thick, or angles less than $2'' \times 2'' \times \frac{1}{4}''$.

Joints and connections

185. The next point to be considered is the method of joining sections of materials together to form a structure. These being made of limited dimensions in length, breadth and thickness, it may be necessary to add to the stock sizes or to combine one section with another of a different kind. For this purpose connecting plates and angles are used which join the different sections together by means of clinched rivets.

186. **Rivets and riveting**—A rivet is made from round wrought iron or mild steel of the best quality having the head in one piece with the body. Before it is fixed, each rivet consists of a small spindle or shank and a head formed like a cup. There are various names given to rivets according to the shape to which the points formed in fixing but the two kinds in most common use are shown in the two figures below:



The cup-ended rivets are adopted for general use, the countersunk variety being only used where projecting heads would be in the way and a plain surface is desirable. For the latter, the hole in plate or bar is countersunk to form the counterpart for the head.

To form a joint, holes are either punched or drilled in the plates or bars it is desired to connect, the rivets are heated to a red heat* in a portable Smith's forge, taken quickly one by one from the forge and inserted into the holes while still red hot. The projecting end is then either rapidly hammered down by hand or pressed home by a machine to form a head similar to that on the other end of the rivet, the head in hand riveting being kept firmly pressed against the plate during the operation by a dolly. In the case of the countersunk rivet, the shank is hammered into the recess formed for it in the plate. For the cup-end rivet, the button is formed by means of a short bar called a "snap" which has one end hollowed out to form a cup. When the projecting red hot end has been hammered down roughly into a lump over the rivet hole the snap is held over the irregular lump and struck repeatedly in different positions by a heavy sledge hammer to form a neat cup head while the iron is still hot and malleable.

A good rivet, well fixed, should be so squeezed into its hole by the process of hammering when red hot that it completely fills the hole between the heads. Loose rivets may be detected, when cold, by tapping between one end smartly with a light hammer and feeling the other end between the fingers at the same time to see if it is firm. An imperfect rivet under this test will be felt to move under the strokes of the hammer and will emit a sharp metallic clink by which it may be easily detected. All imperfect rivets should be cut out and replaced.

187. Rivets are preferred to ordinary bolts with heads and nuts for connecting plates and bars in bridgework because, being hammered close to the faces of the plates, they hold more tightly and their shanks are not so likely to become oxidised as those of bolts; moreover, as rivets are nearly always fixed when hot, they contract in cooling and draw the plates together with great force.

188. Drilled holes are preferable to punched holes, because the process of punching is not so accurate as drilling and it tears and injures the plate to some extent round the edges of the holes, especially when the iron is not of good quality. In really first class work, when several plates have to be riveted together, the rivet holes are of a diameter less than the thickness of the plates, or the quality of the iron is inferior, drilled holes are desirable. For ordinary work, holes in thin plates and

*The head should not be quite so hot as the shank which is hammered down and pressed into the hole to fit it completely. To effect this the rivets are placed in a plate bored with holes so that their heads are on one side of the plate and their tails on the other. The plate is put over the fire with the tails downwards which keeps the heads comparatively cool.

those of a diameter greater then the thickness of the plates may be punched. Punched holes are sometimes formed smaller than required and the rough injured edge is afterwards drilled out by a process called "riming."

189. The aggregate section of the rivets in any joint must be determined by the stress that will come upon them. The method of calculating the strength of riveted joints to meet the stresses in different parts of a structure is fully explained in the College Manual on Applied Mechanics and will not be touched on here but there are certain empirical rules adopted in workshop practice for determining the proportion and pitch of rivets according to the size of the plates and bars connected which it will be useful to note for convenient reference.

Sir William Fairburn's rule for the diameters of rivets in punched holes is as follows:

For plates less than ½ inch thick, the diameter of the rivet should be about double the thickness of the plate. For ½ inch and thicker plates, the diameter should be about 1½ times the thickness of the plate.

When holes are drilled, they may be a little smaller in proportion to the thickness of the plate.

When plates of different thicknesses are joined the rivet is proportioned with reference to the thickest of the plates.

The rivet holes are generally made ⅛ of the diameter larger than the cold rivet to allow for the expansion of the latter when heated before insertion.

Professor Unwin's rule for the diameter and proportion of rivets joining several plates is as follows:

Diameter of rivet$=1{\cdot}2\sqrt{t}$ to $1{\cdot}4\sqrt{t}$ where t=thicknesses of plates in inches.

Height of the head of the rivet should be about ¾ of the diameter of the spindle and the diameter of the head from 1½ times to twice that of the spindle.

The length of the rivet before clinching measuring from the head=sum of thicknesses of plates to be riveted+2½ times the diameter of the rivet.

The "pitch" of rivets, or their distance apart from centre to centre, varies according to the stress upon the joint and the number of rivets required in a given space. A very common pitch for girder work is form 3 or 4 to 6 inches, i.e. about 4 or 5 diameters of the rivets.

No punched rivet hole should be nearer to another than 1 or 1½ diameter (between the edges of the holes) otherwise there is a possibility of breaking the two holes into one; nor should any hole be allowed nearer to the edge of the metal than its own diameter. Drilled holes may be a little closer together than punched holes.

190. **Riveted joints**—These may be of the following descriptions: (1) Lap joints, (2) Cover joints (*a*) single (*b*) double, (3) Angle joints.

Lap joints are formed by riveting together plates that overlap one another as in Figs. 95, 96. The overlap should not be less than 3½ to 3½ times the diameter of the rivets in single riveting or 5½ to 6 diameters in double riveting.

Fig 95

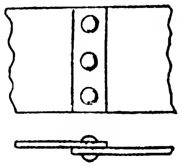


Fig 96

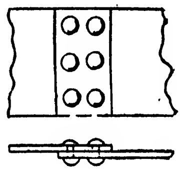


Fig. 97.

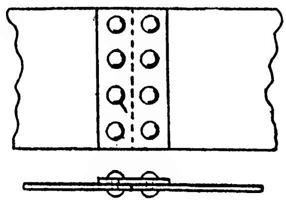
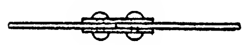


Fig. 98.



Cover joints are those in which the ends of the plates meet one another, the joint being fished either with a single cover plate on one side as in Fig. 97 or with one on each side as in Fig. 98. When two cover plates are used each of them should be of thickness not less than half the thickness of either of the plates to be united.

At joints in compression, the ends of the plates to be connected should butt evenly against one another, but this can seldom be attained in practice as the very process of riveting draws the plates slightly apart and the edges are generally caulked to conceal the gap. As the perfect butting of the plates is seldom or never secured, the whole thrust at a joint in compression is transmitted through the cover plates and rivets, and for the best bridge work it is now assumed that all the joints shall be of sufficient strength to take the whole strain through the rivets.

Single riveting consist of a single row of rivets connecting plates in any form of joint as in Fig. 95; **double riveting** of two rows of rivets as in Figs. 96, 97, and **multiple riveting** of three or more rows. Double or multiple riveting may be of the "**chain**" pattern or "**zigzag.**" The former is formed by lines of rivets in the direction of the stress parallel to one another on each side of the joint as in Fig. 99; the latter by lines of rivets so placed that the rivets in each line divide the spaces between the rivets in the adjacent lines as in Fig. 100.

Angle joints are usually formed by angle irons, each arm of which is riveted to one of the plates or bars meeting at the angle as in Figs. 101 to 105 which represent some of these joints in common use.

Fig.99.
CHAIN RIVETING

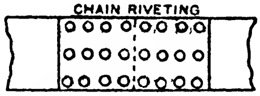
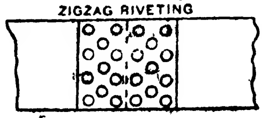


Fig.100.
ZIGZAG RIVETING



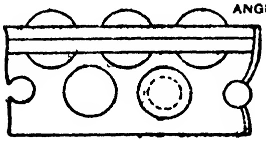


Fig 101
ANGLE JOINT

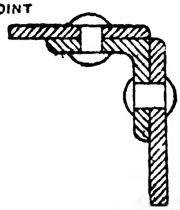


Fig. 102.
JUNCTIONS BY ANGLE BRACKETS.

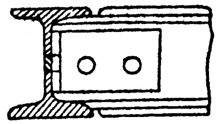


Fig.103.

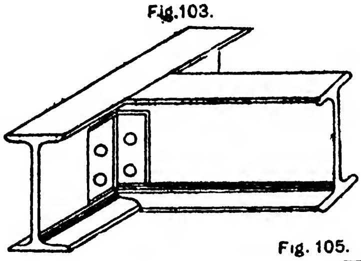


Fig.104.

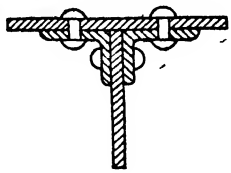
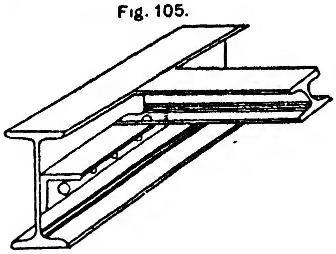


Fig. 105.



191. **The relative strength** of different kinds of riveted joints as compared with that of the solid plate has been found to be approximately as follows :

Strength of solid plate being	..	..	..	..	100
That of the single riveted joint is		..	..	..	56
Double riveted joint ..	..	..	..	..	70
Multiple chain riveted joint	..	..	..	..	85

192. **Causes of failure of riveted joints**—Riveted joints are liable to fail in different ways according to their form and the nature of the stress brought upon them.

The rivets themselves may fail by their heads being shorn off or by the shanks being cut in two by a shearing stress.

Any plate may fail either because its effective sectional area after the rivet holes are cut out is insufficient to resist the stress or by the rivets shearing through the strip of plate beyond them.

If the rivet is not sufficiently large it may not provide sufficient bearing area and either indent or crush the plate round edge of the hole or be indented and injured itself by the plate, causing a loose joint.

193. For important bridge work it is advisable to calculate carefully the stresses which will come upon a joint and to arrange the number, size and position of the rivets, their distances apart and the dimensions of the plates accordingly. Such calculations are, however, beyond the scope of this Manual and will be found fully explained in the Mauual on Applied Mechanics. See Appendix D.

Types of iron and steel bridges

194. The different component parts of the structure of iron and steel bridges and the methods of putting them together having been fully explained in the above paragraphs, we will now consider the different types of such bridges commonly adopted in ordinary Engineering practice.

They may be classified broadly as follows :

(*a*) Suspension bridges.
(*b*) Arch bridges.
(*c*) Bowstring girder bridges.
(*d*) Trough plate bridges.
(*e*) Rolled joist bridges.
(*f*) Plate girder bridges.
(*g*) Tubular or Box girder bridges.
(*h*) Lattice or braced girder bridges.
(*i*) Continuous girder bridges.
(*j*) Cantilever bridges.
(*k*) Movable bridges.

Each of these types will be briefly described below. There are so many different varieties that it will be impossible in this Manual to go into minute details regarding the design of each type, but examples have been referred to in most cases and the books in which they are to be found, so that students requiring more detailed information regarding any particular type will know where to find it.

195. **Suspension bridge**—This type of bridge is generally adopted on hill roads in India as it consist of very light members which are easily transported. It is hardly ever used in the plains Details of its construction and erection have been given in the chapter on Temporary Bridge

Fig. 106.—ST. LOUIS BRIDGE, MISSISSIPPI RIVER.

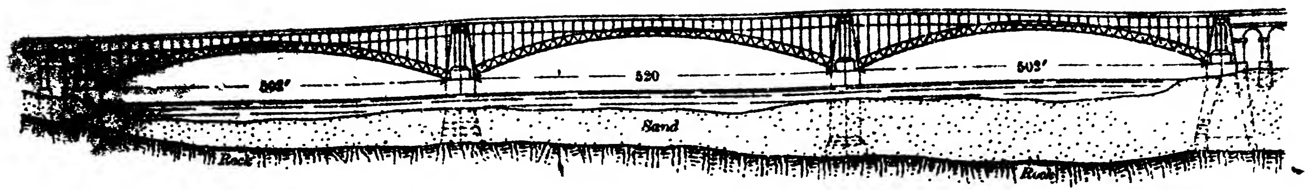
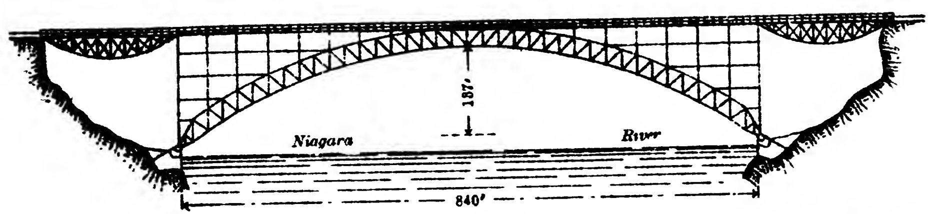


Fig. 107 —NIAGARA FALLS AND CLIFTON BRIDGE.



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and it will not be further referred to in this chapter. See the examples of permanent Suspension Bridges in England and America quoted in paragraph 63.

196. **Arch bridges**—The capacity of iron and steel to withstand tension as well as compression which dispenses with the necessity of keeping the line of resistance within the middle third of the thickness, and their lightness in relation to their strength, as compared with masonry, render these materials most suitable for arches of large span.

Cast iron arches in ribs were formerly built for large spans, but for the reasons given in paragraphs 43 and 183 cast iron is not now employed to any extent for bridge building. Cast iron arches will not therefore be further referred to in this chapter.

There are numerous examples of arched bridge of wrought iron and steel of very large spans recently built over deep gorges or rivers in which pier foundation would have been very expensive. Two examples of these, fairly typical of the rest, are given below.

The St. Louis bridge, carrying a railway and an over head roadway across the Mississipi River in America, was built in 1867-74 with a central arch of 520 feet span and a rise of 47 feet and two side arches of 502 feet span with a rise of 43⅝ feet. It was the first bridge of large span constructed of steel. Each span of the bridge consists of four arched ribs composed of two steel tubes, one above the other, 12 feet apart centre to centre and connected by triangulated lattice work. Each tube has an external diameter of 13 inches and increases in thickness from 1·16 inch at the crown to 2·5 inches at the springing. See Fig. * 106.

An arched steel bridge of the very large span of 840 feet was erected across the Niagara River in America a short distance below the famous falls in 1898. The arches spring from the rock some distance down the sides slopes which are spanned on each side by girders of about 200 feet span. The rise of the arch is 137 and the depth of the arch at the crown 26 feet. See Fig. 107.

A metal arch tends to rise and fall slightly with variations of temperature. When the arch is fixed at its springing, as the St. Louis bridge, the stresses due to change of form have to be borne by the elasticity of the metal. These stresses are greatly reduced by leaving each rib free to rotate slightly on its bearings. The required hinging at the springs for this purpose has been provided in the more recent arched bridges of large span by placing the ribs on cylindrical steel pins

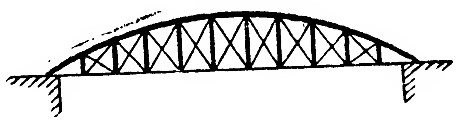
* "Civil Engineering" by Veron Harcourt. By permission of publishers.

at the springing or by tapering down the ribs to pivots at the springings, or, as in the case af the Niagara bridge, by making the ribs bear on a curved surface at the springings on which it is left free to move.

The facility with which large metal arches can be built out from the abutments is one of the chief advantages of this type of bridge when scaffolding is impossible in a deep gorge or a rapid stream. See paragraph 227.

197. **Bowstring girder bridges.**—This form of girder is really a metal arch in which the thrust is borne by a horizontal tie joining the two ends which takes the place of the string of a bow. The tie makes the arch a self-contained structure, which does not require heavy abutments to bear the thrust. The roadway in such bridges is usually suspended from the arch by means of vertical rods which are braced diagonally to provide for the unequal distribution of moving loads. See Fig. 108. This type of girder is suitable for very large spans and is now rarely used.

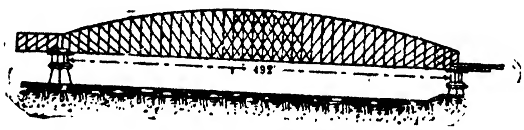
Fig.108.



198. **Parallel flange girder bridges.** – We come now to the type of girders in common use, viz., those with parallel flanges in which the bending moment is resisted by the upper and lower horizontal flanges and the shear by a vertical plate web or open lattice work composed of struts and ties. See plates A and B, appendix D. In describing these girder bridges it will be assumed that the student is already acquainted with the theory of transverse strain. This theory is fully treated on mathematical principles in the College Manual of Appplied Mechanics. He is recommended to master throughly the methods of calculation of stresses and strains in iron structures before proceeding to study the practical details of bridge building. He should recognize clearly that to render a structure secure, a quantity of material proportional to the stress must be provided in every part, and economy requires that the material should nowhere exceed that proportion. A bridge will not under actual conditions be uniformly strained everywhere but it should be so designed and calculated that there will be uniformity in the **intensity of stresses** of the different parts under all the varying conditions to which it will be subjected.

199. The bending moment of a uniformly loaded girder is a maximum in the centre of the span and diminishes towards the piers. Consequently the sectional area of the flanges must be made largest in the centre of the girder and it may be gradually reduced towards the ends by diminishing the number of plates, or, the sectional area of the flanges being maintained, the depth of the girder may be reduced in proportion to the reduction in the bending moment by curving the top flange. For short and moderate spans the former system is adopted, the flanges being made exactly parallel, but, for very large spans, over 250 feet, the modern practice is to adopt a combination of the two systems for economy in design. See Fig* 109.

Fig. 109.



200. The shearing stress of a girder uniformly loaded, on the other hand, increases regularly from zero in the centre of the span and reaches a maximum over the points of support where it amounts to half the distributed load. The web plates accordingly of plate girders have to be strengthened and stiffened towards the ends by verticle T or ∠ iron stiffeners and gusset plates while the struts and ties of open girders are made successively stouter from the centre of the span to the abutments.

201. The web of a plate girder may be regarded as a continuous series of lattice bars subjected both to compression and tension and transmitting the stresses to the flanges at every point of their length. Web plates are serviceable for small girders where lattice bars suited to the strain would be very small, their workmanship relatively costly, and their connections with the flanges somewat inconvenient; but these plates cannot be exactly proportioned like lattice bracing to the exact stresses they have to resist at each point and the vertical T or ∠ iron stiffeners, usually introduced, only roughly provide for the increasing compressive stress towards the ends so far large spans, the lattice girder is the more economical design of the two. The ordinary practice is to use plate girders for spans not exceeding 60 feet and lattice girders for spans of 60′ and upwards. The depth of girders for vertical stiffness should be $\frac{1}{10}$ to $\frac{1}{15}$ span, generally $\frac{1}{12}$. The width of horizontal flanges for lateral stiffness should be $\frac{1}{35}$ to $\frac{1}{50}$ span. Examples are given in plates A and B, App. D of plate and lattice girders which show their details of construction.

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202. For small road bridges up to 15 feet span, **trough plates** are commonly used to span the opening as shown in Figs. 110 to 112. The troughs are laid longitudinally and tied across with a tie bar at the centre of the section attached to the under side of each trough and to the side plates or channels forming the kerbs. At the ends they are tied across by bearing plates 12″×½″ which rest on bed plates of the same size secured to the abutments. The bed plates generally project about 6 inches beyond the bearing plates and have strips riveted on at the projections to prevent lateral movement of the trough plate structure.

203. For railway bridges up to 20 feet span and for road bridges from 15′ to 20′ span, **rolled steel joists** are usually adopted as girders. In the former, the joists are placed immediately blow the rails and tied across for lateral stiffness by iron rods or bars. The sleepers are laid across the joists and the rails are carried by the cross sleepers in the ordinary manner. In road bridges the roadway is carried by trough or buckle plate flooring on the joists or by shallow brick arches called "Jack arches" thrown between the lower flanges and having their haunches filled up with concrete. Metalling of the required thickness is laid over the flooring so formed. In the process of rolling, neither the width nor thickness of the web and flanges of a rolled joist can be altered to suit the varying strains, but, for a span not exceeding 20 feet it is of no consequence, practically, whether any of these dimensions are varied or not and the rolled joist has the advantage of cheapness, simplicity and freedom from all joists and rivets.

204. **Plate girders** are generally employed on roads and railways for spans between 20′ and 60′. On railways, they have sometimes been used up to 80 feet. They consist essentially of a web of plate iron and flanges of two angle irons, one on each side of the web, with or without horizontal plates riveted to the angle irons. See Appendix D. For the smaller spans angle irons only will be found sufficient in the flanges, but for larger spans flange plates are required in addition to the angle irons to resist the compressive and tensil stresses.

In designing the section of the flanges, it must be remembered that a proportional addition must be made to neutralise the loss of strength (about 20 per cent.) due to the rivet holes in both flange plates and angle irons. In these girders the section of the web can be proportioned to some extent to suit the shearing strains by using thinner plates at the centre and thicker towards the ends; the upper and lower booms can be likewise proportioned by the use of thicker plates in

PART ELEVATION

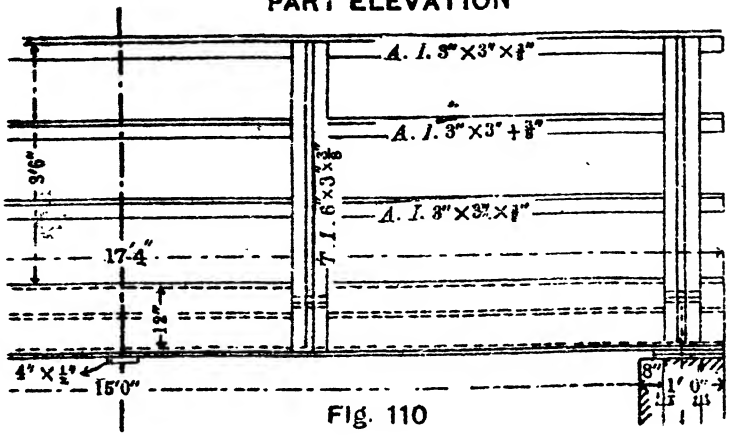


Fig. 110

Fig. 111. CROSS SECTION.

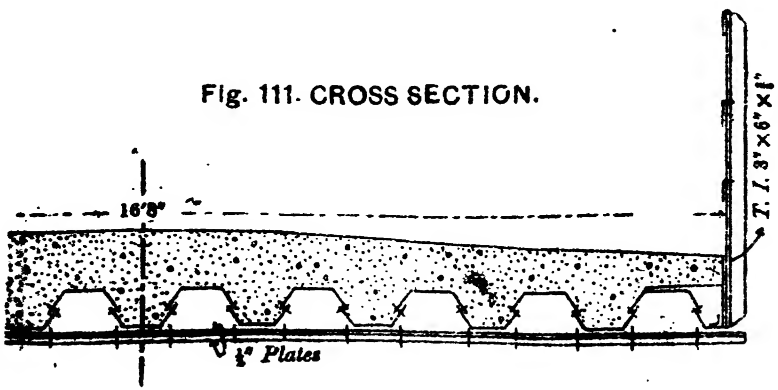
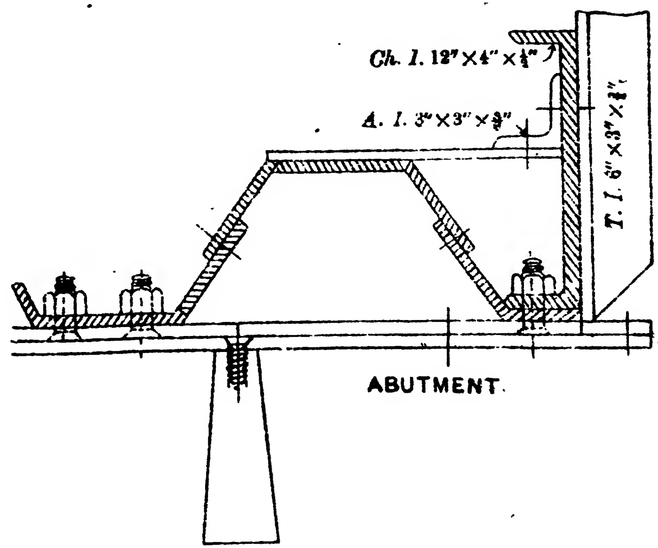


Fig. 112.



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the flanges at the centre, or, using the same thickness of plate throughout, by gradually increasing from the ends to the centre the number of plates superimposed one on the other in building up the booms. In designing the booms, the following points should be attended to :—(1) The size of the plates should not if possible exceed the limiting sizes kept in stock by manufacturers as a higher rate is usually charged for larger plates. There is no precise rule for these sizes. They vary from time to time and should be ascertained definitely before the design of a large bridge is undertaken. (2) The plates should break joint at connections so that no two joints occur in the same cross section, the minimum distance between two joints being not less than twice the pitch of the riveting at the joints. (3) The plates should be uniform in dimension as nearly as possible and the riveting so arranged that the pattern for each plate is as nearly as possible the same. Any departure from this rule involves risk of mistakes by workmen.

The webs of plate girders consist of vertical plates of the full depth placed end to end and connected by T irons on both sides which act as covers and at the same time stiffen the web plate to resist the diagonal compressive forces to which the shearing force gives rise. By their transverse resistance to flexure the T iron stiffeners not only oppose the tendency of the web to buckle but they also, in large bridges with cross girders, transmit the weight of the cross beams through the web plate uniformly to both booms. When cross girders are used there should be a T iron stiffener at the junction of each cross girder and intermediate stiffeners of less section if necessary. Since the shearing force is a maximum at the abutments and a minimum at the centre it is a good plan to design the web first at the ends and then at the centre for which the plates, in small bridges, are generally not less than ¼ inch and for the larger spans nor less than 3/8 inch. It will then be easy to graduate the section ; examining the strength at those points only at which the section changes.

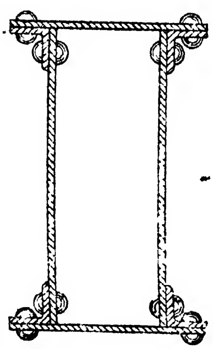
The web at the ends of a girder where it rests on a pier or abutment is to be considered a simple pillar which receives and distributes to the web the entire vertical reaction at that point. The web therefore requires an addition to its thickness over the points of support and an increase in the number of T irons riveted to it.

In road bridges the flooring consists generally of trough or buckle plates covered with metalling. In railway bridges it is not usual to provide any floor over plate girder bridges.

Diagonal cross bracing at intervals is required between the main girders for lateral stiffness and to resist wind pressure.

205. **Tubular or box girders.**—Are only varieties of plate girders. A tabular girder consists practically of two plate girders, the top and bottom flanges of which meet overhead and down below. The first girders of this kind were erected in 1848 across the river Conway for the Chester and Holyhead Railway. The Conway bridge has a clear span of 400 feet and consists of two parallel tubes containing the up and down lines, placed a clear 9 feet apart, 15 feet in width, and increasing in height from 22½ feet at the ends to 25½ feet at the centre. The Conway ends of each tube were fastened down to the abutments but the other ends rest upon rollers and gunmetal balls to allow for expansion and contraction due to variations of temperature. Another celebrated bridge of this kind is the Britannia Tabular bridge on the same railway across the Menai Straits which has two central spans of 459 feet and two side spans of 230 feet.

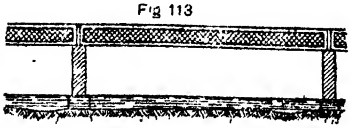
Small tubular girders are commonly called **box girders.** They are formed by four plates connected by angle iron as shown in the margin. They should be at least large enough to admit a man or a boy so that they can be painted periodically inside to prevent corrosion.



Tubular girders of a cellular shape, though very suitable in form for resisting compressive stressess, are with difficulty protected efficiently against corrosion and involve the use of an excessive quantity of metal in the double solid plate webs. They have now been abandoned in favour of open web girders with diagonal bracing proportioned to the strains in the different parts of the girder which are considerably lighter than solid web plates and offer much less surface to the wind.

206. **Latice girders**—Are of various types but the most common form known by this name is that shown in the figure on the margin. Formerly, a common type of construction for lattice grider bridges of large span consisted of parallel top and bottom flanges of plate iron closely connected by numerous diagonal bracing bars which helped to

Fig 113



stiffen the girder and enabled the dimensions of the lattice bars to be kept very moderate as illustrated by the sketch on the margin. More recent practice in large bridges of this type is however in favour of resorting to greater depths for the girder in proportion to their span and placing the struts vertical, thereby reducing their length as "pillars' under compression whilst retaining the diagonal position of the ties. See Fig. 109. The reduction in depth of a large girder towards the ends as exemplified in the sketch has the advantage of reducing the length of struts and ties where the stresses on them are greatest. The proportion of depth to span in lattice girders of very large span is now 1/12th to 1/10th in the middle.

The cross bracing shown in the central portion of Fig. 109 is, as the student is no doubt aware, put in to counteract the changes of stress in the central bays due to the action of travelling loads. In the ordinary diagonal lattice girders the central tie bars are constructed similarly to the struts in order to adapt them for resisting the compressive stresses which unequal loading is liable to impose on them. It may be useful to note here that in braced girders under uniform load the bars which incline towards the abutment are usually struts and those which slope from the abutment towards the centre are ties.

In the earlier lattice bridges, as stated above, a large number of bracing bars were employed cutting up the web into very small reticulations, the bars being riveted together at every intersection to approximate to the condition of a plate web. The riveting of struts to ties at frequent intervals undoubtedly stiffens them against flexure in the plane of the girder, but at the same time the multiplication of their number and consequent reduction of their individual dimensions weakens them against transverse flexure and enhances the proportionate waste of material towards the centre where the shearing force vanishes, for, in general, they cannot practically be reduced below certain fixed dimensions. In the latest and best practice, therefore, the number of triangles in the bracing is reduced as much as possible, the limit being always the practical conditions which fix the maximum size of a single strut.

In girders of very large span, their dead weight forms a large part of the total load and it is therefore very important to keep it down as low as possible. This is effected chiefly by increasing the depth of girders as above explained and by using a good quality of steel instead of iron. Owing to the greater strength of the former, not only are the sectional areas and weight of the different members of a girder reduced in proportion to its strength as compared with that of wrought iron but a further reduction becomes possible owing to the diminution in the dead weight of the girders themselves which they have to carry. In America, moreover, a further attempt is made to reduce the dead weight of girders by making the bottom flanges of a series of bars, the bracing being joined to these and to the top flange by bolts or pins. The top flange, being under compression, is generally of trough section TT and consists of plates riveted together. This section, though suitable for a member under compression, is considered to be somewhat wasteful for the bottom flange in tension owing to the loss of area due to rivet holes and the additional weight of cover plates over the joints; and consequently economy of material is obtained by the use of simple eye bars connected by bolts or pins. For the same reason, the use of pin connected bars for the tension bracings instead of riveted plate ties reduces the amount of metal. An economy of about 27 per cent. has been gained in some cases quoted by Claxton Fidler in his treatise on Bridge Construction by adopting these modifications. In Europe, however, it is the ordinary practice to make the lower flange the same section as the upper and to build up the tension members of riveted plates because the vibration of heavy rolling loads has been found to loosen the connecting bolts and pins and the failure of any one joint would seriously imperil the safety of the whole structure. Combining, however, as they do the important elements of economy, portability and simplicity of construction, pin connected girders might sometimes be used with advantage for road bridges, even if they are considered unsuitable for large and important railway bridges for the reasons stated.

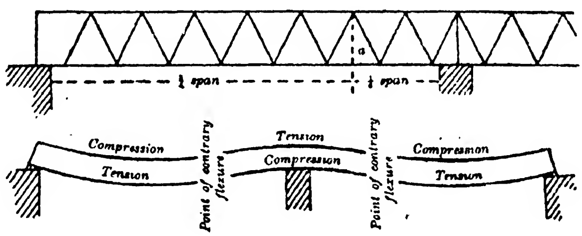
To resist wind pressure and for lateral stiffness, diagonal cross bracing is fixed at intervals between girders when these are below the roadway. When the roadway is carried on the lower boom, overhead counter bracing is necessary between the upper booms.

The different types of lattice girders commonly adopted in America are distinguished by the names of the persons who first introduced them, e.g. the Howe, Whipple, Linvile, Pratt trusses. These exhibit certain variations in the arrangement of the bracing, but they are all essentially lattice girders and will not be further referred to.

207. **A continuous girder** covers several spans resting on intermediate supports and, under these conditions, it acts in some parts as a cantilever and in others as a supported beam: and under moving loads, in some parts it acts as either alternately. Such continuity is generally attended with great advantage as far as economy of materials in the girders is concerned, especially in those with symmetrical cross sections and under steady loads. There are cases, however, when it is

disadvantageous. The piers required for their support have to be considerably stronger than those for disconnected girders as they carry a much greater proportion of the load. Continuous girders have often been resorted to for a series of moderate spans but they are not suitable for spans under 150 feet nor have they been extensively adopted for very large spans owing to the intensification of the stresses which would occur in parts of the girder if the slightest settlement took place in one of the piers and also on account of the difficulties experienced in their erection when the size of the girders and the width of the spans preclude the possibility of building them over one abutment and rolling them out over the piers. See Fig 180.

In a girder of this description extending over two spans under steady load, the portion next the abutment for a length of three-fourths of one span may be treated as a detached and independent girder with the loading due to its length, the remaining one-fourth being treated as a cantilever projecting from the central pier and carrying at the end one-half of the load of the supposed separate girder covering the rest of the span as well as its own load. The point "A" where the supposed girder and cantilever join is usually called the point of contrary flexure because the curve assumed by a loaded continuous girder changes at this point from concave to convex.

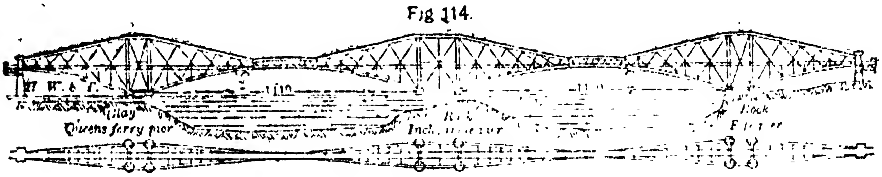


In a continuous girder of three or more spans and when the spans are unequally loaded, the position of the point of contrary flexure is not that stated above but varies with the moving or unequal load and is different in different spans. The method of calculating the stresses of continuous girders under varying conditions will be found fully explained in the Applied Mechanics Manual.

208. **Cantilever.**—As applied to bridges, cantilevers consist essentially of girders projecting like brackets from the piers which support them and either stretch across the span themselves or cover a portion of the span on each side and support at their ends independent central girders whose span is in this way considerably reduced.

The most distinct form of cantilever bridge is that in which symmetrical arms project on each side of the piers, the arms over the main span supporting a central girder and the outer arms towards the abutments, stretching over smaller side spans, being anchored down to the bank on each side or provided with additional load in the end pillars to counterbalance the weight of the central girder with its load. This design has been adopted in the Niagara and the Forth bridges. The latter is shown in Fig.*114. It will be seen in this case that as the central cantilever cannot be anchored down the unequal load caused by moving trains has been provided for by enlarging and strengthening the central part of the structure over the pier. It will further be noticed that the cantilevers and their high supports over the piers are stiffened laterally and strengthened against wind pressure by widening them considerably towards their base and especially over the piers.

Fig 114.



The steel cantilever span of the Sukker bridge in the Punjab is remarkable for having only a single span and for the height of its cantilevers which is about one-fifth of the span. See Fig †115. The span is 790 feet between abutments consisting of two single cantilevers projecting 310 feet from their supports on each side and carrying a central girder span of 200 feet on their inner ends. The cantilever being unbalanced by shore spans, this form of bridge for a single opening seems uneconomical and was only adopted apparently on account of the unusual length of span and because it was found impossible to maintain staging in the river for a period long enough to admit of the erection of an ordinary trussed girder bridge. With such excellent rock abutments as those available at the site, it appears, however, open to question whether a braced steel arch bridge would not have been a more suitable design: it could have been erected by overhanging from the shore ends and it would probably have been more economical.

In bridges of very large span, the dead load becomes more important than the moving load as the span is increased, and this increase reaches

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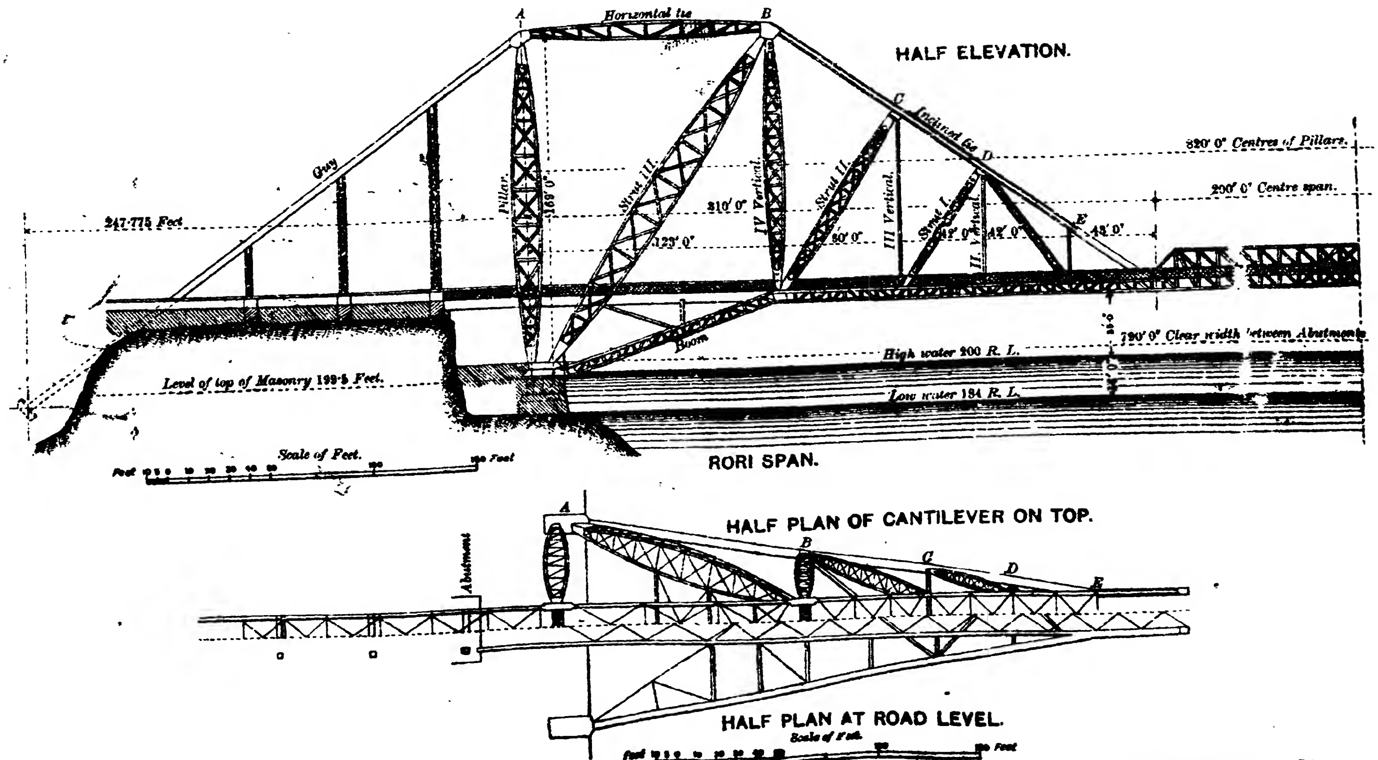
†Proceedings, Inst. C. E. Vol. CIII.

Fig. 115.



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a limit beyond which it is impracticable to design bridges of any particular type because they will not bear their own weight rather than on account of the strains due to the relatvively small moving load they may be required to carry. This shows that those types of bridges are best suited for very large spans which concentrate their greatest weight of metal near their supports, like cantilevers, rather than those which are liable to be heavier near the centre of the span as in the case of ordinary girders. Moreover, symmetrical double cantilevers are well adapted for very large spans on account of their self-contained and balanced structure and their suitability for convenient erection by building out from their supports as shown in Figs. 133. 134.

209. **Movable bridges.**—When a road or a railway crossees; a navigable river, canal or dock, it is necessary to provide for the passage of masted vessels or steamers by making the structure movable. There are three distinct types of movable bridges which will be only briefly referred to here as they are but seldom required in India. References have been given in each case to papers from which full information can be obtained, if necessary—

(*a*) **Swing bridges.**—These have balanced girders swinging round a quadrant of a circle horizontally over a pier or a pivot or on a turn table with cast steel rollers. This is a common type. Two good examples of it are the Castletown Swing Bridge and the Hawarden Bridge described in the Proceedings Institute of C. E., Volumes XCII and CVIII respectively.

(*b*) **Traversing bridges**—This type rolls backwards and forwards across the opening and is only resorted to when the space above and below the opening is not large enough to take a swing bridge when opened. The girders have counterpoised tail ends exactly the same as those of a swing bridge and can be raised from their supports. When raised (by a hydraulic ram as a rule) they are drawn back on rollers along one of the approaches; leaving the opening clear. See the traversing bridges at the Mill wall Docks, London, and at the Barry Docks described in Proceedings I.C E., Volumes L and CI respectively.

(*c*) **Bascule bridges.**—The girders of these bridges move up vertically on a horizontal hinge and assume an upright position when the bridge is open. For large spans, the bascule or moving part is made to open in halves like the two leaves of a door, each half working upwards on a hinge on an abutment. The greater part of the weight of the bascule is counterbalanced by a dead weight and gearing is provided by which it may be readily raised or lowered by hand or hydraulic power. The finest example of this type of bridge if the Tower bridge of London with a span of 200 feet which will be found fully described in "Engineering." Vol. LIX, page 7.

210. **Roadway.**—The chief types of girders and the main details of their construction having been described, a few remarks are now necessary on the roadway with its cross or longitudinal girders carried on the main trusses.

211. In roads bridges of large span the main girders are usually spanned by cross girders and the latter are covered by buckle or trough plates on which the metalling is laid. See paragraphs 202 to 204 for description of roadway of short spans. Whin lattice girders are used for road bridges, they are usually utilised as railings and the cross girders rest on the lower booms or are suspended from them by riveted joints. Tarred planks are sometimes used for economy over the cross girders instead of iron plates, but these are of a perishable nature and should only be used when the funds available will not admit of the use of the more substantial material.

212. In the earlier railway girder bridges, cross beams were used to carry the roadway placed at sleeper distance, or about three feet apart. See Figs. 116, 117. Over these were placed the longitudinal timbers supporting the rails. It was assumed that these timbers were strong enough to distribute the load over the cross girders, and the cross beams, on this supposition, were calculated to carry the same average load as the main girders. Some bad accidents occurred and aroused the attention of the Engineers to the fallacy of this assumption. The weight of a locomotive is not carried equally by all its wheels and the greatest weight on any pair of wheels is brought in turn over each girder. The longitudinal timbers decay in time; they are not continuous but have joints and joints in the timber and rail may occur over a cross girder; it is therefore possible that cross beams may sometimes have to carry the whole or almost the whole load brought on it by the engine wheels. It soon became evident that the platform would be lighter and more efficient if the cross girders were pitched further apart and longitudinal girders were placed under each rail, as shown in Figs. 118, 119, instead

of placing the cross beams at sleeper distance apart. This plan is now adopted in all largo railway bridges.

Fig 116.

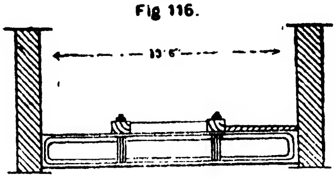


Fig 117

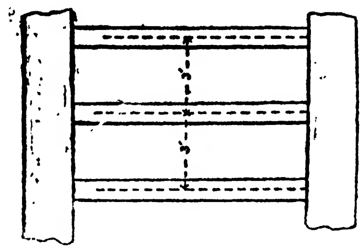


Fig 118.

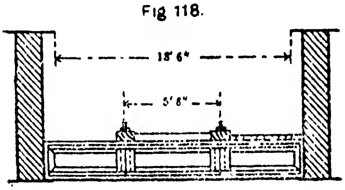
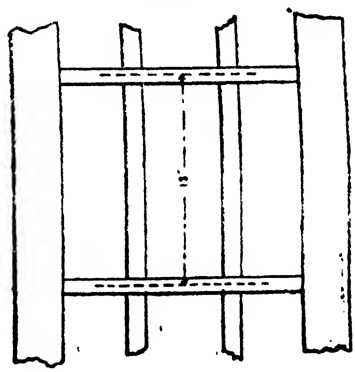


Fig 119

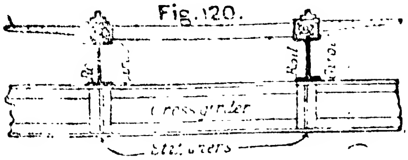


213. The platform in railway girder bridges may be above, between or underneath the main girders. When there is no restriction as to headway under the bridge, it is best placed for moderate spans over the main girder, because the load is then transmitted fairly to the centre of the girders and the span of the cross beams may be reduced with a corresponding reduction of the weight, but this arrangement is obviously unsuited to long spans on account of the narrow " Bridge Base " which results, especially on narrow gauge lines, the " Bridge Base " being determined by the gauge of the line.

When the road way platform is carried on the lower boom, the most economical arrangement, if sufficient headway is available, is to place the rail girders on top of the cross girders and carry them continuously from end to end of the bridge with as large cross girder spacing as may be found to be the most economical and suited for the general design. With this arrangement, the cross girders should be furnished with vertical

T-iron stiffeners on each side of the web or bracing immediately below each rail girder to transmit the load to both flanges. See Fig. 120.

Fig. 120.



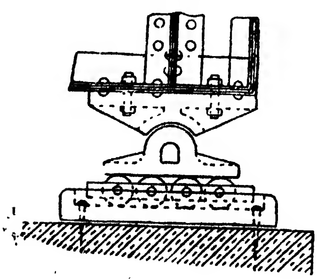
If headway is limited and it is necessary to economise it as far as possible the rail girder may be introduced between the cross girders as shown in Figs. 118, 119.

214. Cross girders on a large bridge should always cross the main girders at the joints of flanges and bracing and the flanges of every large flanged girder should be connected by stiffeners at the ends of every cross girder to transfer the load alike to both flanges.

215. **Support of Girders.**—The weight of a large girder with its load is so great as to require special arrangements for the distribution of the pressure over the supports. The bearing surface on the supports should be large enough to reduce the pressure intensity to a limit well within the safe crushing stress of the material ef the supports. For this purpose, it is usual to attach, by countersunk rivets, a wide thick *bearing* plate of steel to the underside of the lower flange at the ends and rest this on similar thick cast steel *bed* plates fixed to bed blocks of stone or cement concrete built into the abutments. To allow room for expansion and contraction of the girders from changes of tepmerature, one end of each girder is firmly fixed to its support while the other is bolted down to the *bed* plate and bed block through slotted holes o in the *bearing* plate which allow some play for the longitudinal movement of the girders. Instead of this sliding arrangement, the free end of the girder is sometimes carried on cast steel rollers.

To further allow of slight movement in the end pillar vertically when the girder is deflected and to keep it bearing normally on its support, a "rocker" end is often provided in large spans and sometimes the rocker and roller are combined as shown in Fig. 121 on the margin. In the latter arrangement a "saddle block" is bolted to the underside of the girder which rocks on the saddle. The saddle in its turn moves on the set of rollers which themselves roll on the smooth and polished bed plate, the latter being securely bolted to the bed blocks of the pier or abutment.

Fig 121



216. **Loads on girders. Dead load.**—The dead or permanent load of a bridge enters into the calculation of every design from the outset and it is therefore necessary to assume what this weight will be as a preliminary. It may be obtained approximately by comparison with the weight of similar bridges previously constructed and various rules have been formulated for arriving at this approximation.

Whichever rule is adopted, it is very necessary in getting out the final design to revise the calculations and correct the stresses if the assumed weight is found to differ appreciably from the actual, until a sufficient degree of accuracy has been obtained.

For ordinary road bridges, the weight of the platform, which includes cross girders and buckled or trough plate flooring, may be assumed to be 32 lb. per square foot of roadway. For spans under 40 feet, having the longitudinal girders under the flooring, the combined weight of main girders and flooring may be taken to be 800, 900 and 1,000 lb. per lineal foot of span for 12, 14 and 16 feet width of roadway respectively. For spans from 40 to 200 feet, and 16 feet roadway, the weight per foot run of the two main girders (without the platform) may be assumed to be $\left(\frac{\text{Span}}{500} + \frac{\sqrt{\text{Span}}}{100}\right)$ for 14 feet roadway $\frac{9}{10}$ of this, and for 12 feet $\frac{8}{10}$. For bridges of very small spans in which trough plates only are used, the weight is determined in each case from the section and weight of the plates adopted. If mild steel is used the weights should be 20 per cent. less than that found by these rules.

The dead load to be assumed for railway bridges should be those given in the following table which are based on the actual weights of bridges recently made for railways in India, the material being mild steel.

Clear span	Weight in tons per lineal foot of span for single line bridges	
	5' 6" gauge	Metre gauge
10	0·12	0·10
20	0·15	0·13
40	0·25	0·22
100	0·72	0·70
200	0·90	..
300	1·50	..

217. **Live load.**—As the live load on road bridges ordinarily bears a small proportion to the dead load and it moves slowly, it is usual to allow for it, as in the case of arch bridges, by adding 150 lb. per square

foot of roadway of the weight of a steam roller (15 tons), whichever is greater, to the distributed dead load. But in railway bridges, the live load consists of heavy locomotives and trains in rapid movement which produce much more injurious strains than an equal quiescent load and it is therefore necessary to make special provision for it.

The live load on a railway bridge is taken as the weight of two locomotives and a train of carriages. The greatest load is that of the locomotive which in a 5 feet 6 inch line comes on the wheels 7 to 8 feet apart. A further allowance has to be made for the dynamic effect of the locomotives passing rapidly over the bridge, and a greater rate of load must be assumed for short spans than for long ones because short spans are usually covered by one or two locomotives only, while in long spans these are followed by a lighter train and the average load is therefore less in the latter than in the former. As the weight of locomotives and carriages is constantly changing, no fixed rule can be made for live loads on railways, but the rules at present prescribed by the Government of India for single line main girders to suit the heavy locomotives now in use are as follows :

Span in feet	5′ 6″ gauge	Metre gauge
	Tons per foot run of bridge	Tons per foot run of bridge
10	4·6	3·0
20	3·6	2·2
40	2·7	1·7
100	2·0	1·3
200	1·6	1·0
300	1·4	1·[illegible]

The cross girders of a railway bridge platform either receive the load directly from the sleepers or from longitudinal girders under each rail. In the former case they must carry the maximum load on an axle of a locomotive which, in a 5 feet 6 inch line, is 16 tons, and in a metre gauge 8 tons. In the latter case, when the cross girders are more than 8 feet apart in a 5 feet 6 inch line the total load should be taken to be $2\frac{1}{4}$ tons per foot run of distance between cross girders, and for metre gauge, when the distance between cross girders is more than 4 feet, they should be designed for 2 tons per foot run of their distance from one another.

218. In very large bridges, it is further necessary to take into account the effect of wind pressure which acts horizontally across the structure and produces compressive stresses on the booms of the windward girder and tension on those of the leeward one, or, it may tend to cause the bridge to overturn on the lower chord of the leeward girder, or, in the case of

a very high bridge, it may tend to overturn the pier endwise. This effect is increased by the additional surface of a train passing over the bridge. If the bridge be on a curve the centrifugal force of passing trains will also produce lateral strains on the booms. The combination of these extraordinary stresses becomes somewhat complicated, and need only be considered for very large bridges. Wind pressure may be taken to be 50 lb. per square foot on exposed bridge or train surface.

219. **Erection of girders.**—Before proceeding to describe the various modes of erecting girders, it will be useful to consider the method of accurately measuring the span for fixing the anchor bolts of bed stones and bed plates in their exact positions on piers and abutments to receive the finished girders.

220. In small spans this can be done with sufficient accuracy with a steel tape but it is not practicable to obtain a very accurate measurement of large spans in this way. For the latter, the measurement is made by stretching a piano wire of the required length under the same tension and at the same temperature across the different spans to be measured. To compensate for variations of length due to temperature, it is necessary to first fix a standard base line of the exact length and to adjust the marking on the wire afresh every time it is used by stretching it over the base line under a certain fixed tension. The temperature being thus compensated, it is evident that the length of the wire must be constant under the same tension. The required tension is obtained by means of a constant steelyard, the knife edges of which are moved till the pull of the wire exactly balances the bar. The tension should be as great as possible—from 15 to 20 tons per square inch.

221. Spans of 50 to 60 feet, or even 100 feet, are erected with little difficulty on river beds when the height is not excessive. For nine months of the year, the bed of most Indian rivers is dry except for a narrow width which holds the dry weather stream. Under these conditions, the spans may be levelled up with earth successively to form a platform and the girders built up on it on wooden blocks. When the girders have been riveted up, they can be readily lifted into position by means of derrick poles or braced shear legs and ordinary rope tackle worked from anchored crab winches. When, however, large spans have to be dealt with, the height is excessive, or the flow in the river constant, other means must be devised. Generally, the methods of erecting girders may be divided into four classes, (1) erection on staging, (2) erection by floating, (3) erection by pushing or rolling out and (4) erection by overhang of building out from supports. The first two methods are

suitable for erecting single unconnected girders; the third for erecting continuous girders and the fourth for erecting arches and cantilevers.

222. **Erection of girders spanning single openings.**—Erection on staging is the simplest method, and the one commonly resorted to when the height is not excessive and the river is dry the greater part of the year, or, at any rate, not very deep.

223. **Timber staging** will be found economical when only a few spans have to be erected. The construction of the staging depends to a great extent on the design of the bridge and the size and weight of the pieces. A design for this class of staging suitable for truss girders of the Whipple Murphy type is given in Figs. 122, 123. The lower boom (of pin connected bars) is laid on the lower camber blocks and the upper boom (of cellular riveted section) on the upper camber blocks. The erection of the bridge is begun from one end and carried on to the other. When a length of the booms has been put together, the struts and diagonals are connected to these and so the work proceeds until the girder is completed. When both girders of the span have been put together in this way, the overhead bracing above and the platform below are put in. When this is done, the camber blocks are knocked out and the bridge is swung on to its own bearings on the abutments. If the bed of the river is hard and dry, the verticals of the staging may be footed into broad horizontal sills laid on the ground, but, if the bed is soft, piles will be necessary as shown in the figure.

224. **Iron staging** is more economical than timber when a great deal of bridge erecting work has to be done on a road or railway at different sites as it is light and more easily transported. A very simple and ingenious form of this class of staging is shown in Figs. 124 to 127. It was designed and extensively used by the late Mr. Sullivan, Chief Engineer, for the erection of bridges on the Bolan Railway. It consists of a series of light wrought iron tubes braced together diagonally and horizontally. These rest on a platform of rolled iron beams and there is a similar platform on the tops of the pillars. The staging is made in convenient lengths and similar parts are interchangeable so that it can be easily adapted to any length of span or height of bridge.

225. **Floating** completed girders out into position on pontoons has been extensively resorted to where the river to be crossed is too deep for staging. This is done either by floating the girders out bodily on pontoons from the shore where they are put together or by using the pontoons to support part of the weight while the girders are pushed forward from the abutment on which they are put together. Both these methods

TIMBER STAGING.

Fig. 122.

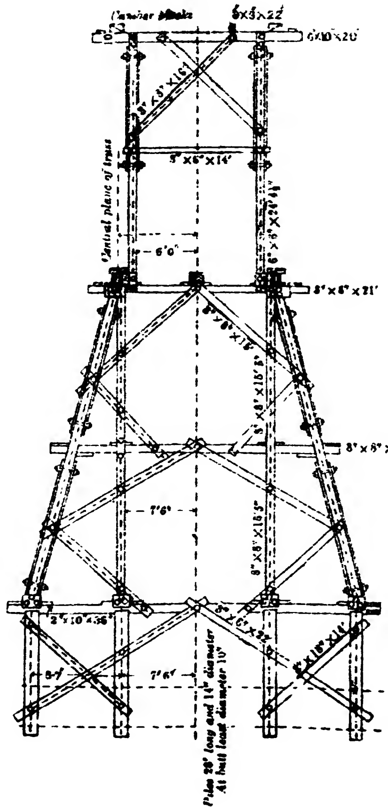


FIG. 123

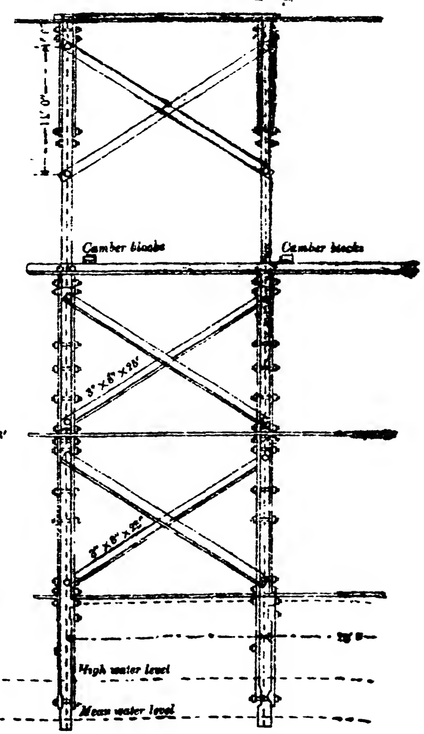


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Fig. 126. PLAN BOTTOM HORIZONTAL STAGE.

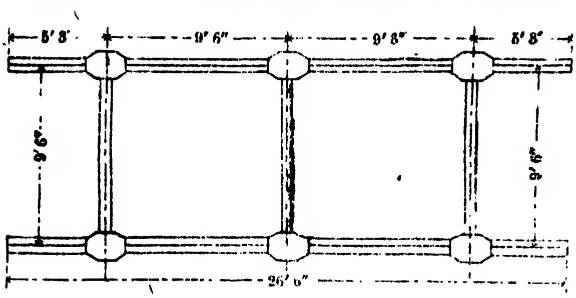

Fig 125.

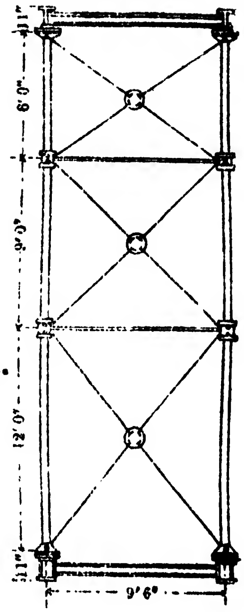

Fig. 127. PLAN INTERMEDIATE HORIZONTAL STAGE.

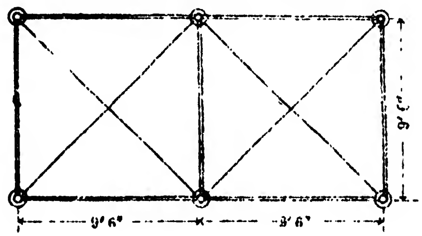

Fig 124. SULLIVAN'S IRON STAGING.

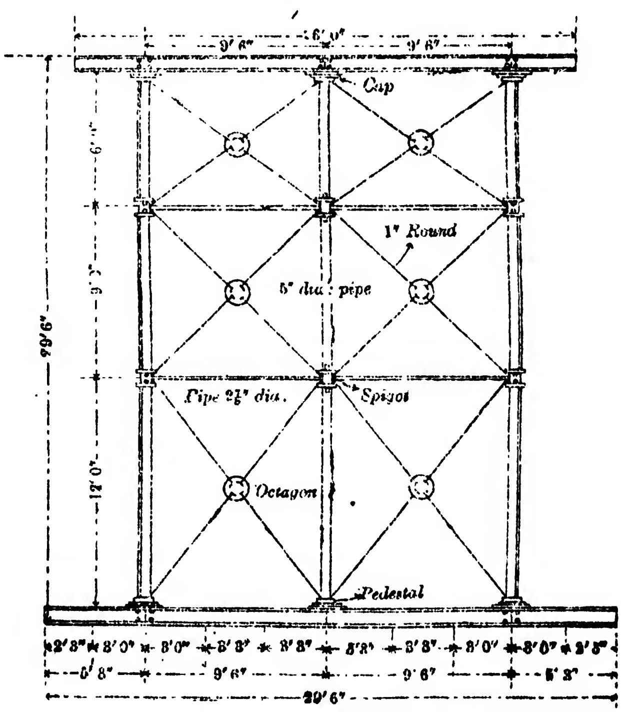

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Fig. 128.

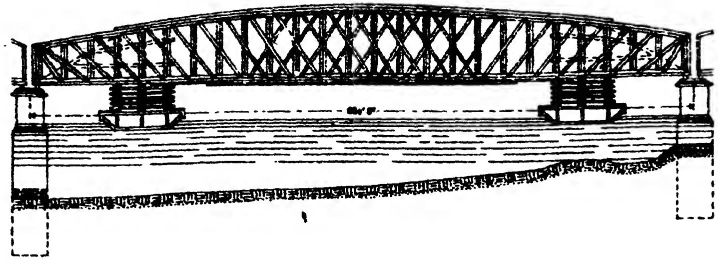
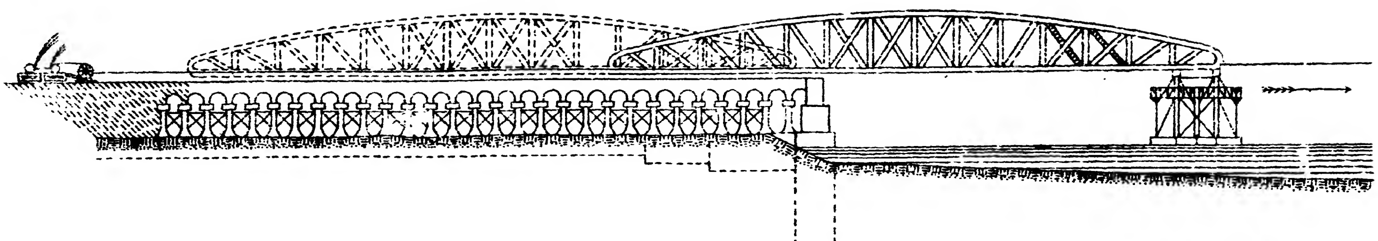


Fig. 129.



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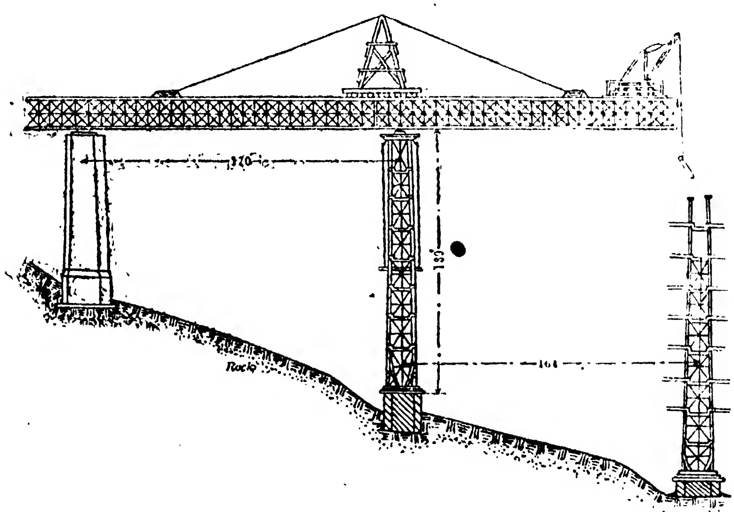
are shown in Figs. 128, 129. In the first* case the girders are usually erected on a high staging on piles projecting from the shore and then taken on two or three flat-bottomed pontoons of the required buoyancy and deposited in their exact position on the piers. In the second case, the usual practice is to erect the girders on the bank on the centre line of the roadway supporting the forward end on a roller bearing. The rear end rests on a substantial trolley. To project the girder forward, chain or steel rope tackle is used which is wound on a capstan by portable engines placed at the rear end. The fore end having been drawn out a short distance from the abutment, the pontoon, on which a stage of the required height has been previously erected, is floated underneath the projecting end and firmly secured to it. It is then floated across the stream with the girder on it to the pier. When the girder is over its bearing on the pier the pontoon is withdrawn.

226. **Rolling out girders** is a system generally adopted with continuous girders. Spans that are not continuous over the supports cannot be erected in this way without elaborate staging to support the weight of the projecting portion or temporary connections between the girders to make them act as continuous girders while they are being rolled out. This system is particularly advantageous where a bridge is designed to cross a river in several spans of moderate size or where a viaduct has to be carried acros a deep gorge. See Fig. 130.

Fig. 130.



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The above figure* illustrates the system of rolling out adopted for a viaduct of considerable height as well as the utilisation of the projecting girder as an overhanging staging from which by the help of a crane at the extremity the tall iron piers were built up.

The only objections to this cheap and simple method of erection are the necessity of stiffening the girder to withstand the special stresses to which it is subjected temporarily during erection quite different from those it has been designed to bear in its final position. These temporary stresses, however, can be reduced considerably by the provision of additional stiffening pieces to strengthen the girder during erection and by supporting the projecting end by steel wire ropes passing over staging erected successively over the piers and anchored to the inner part of the girder.

227. **Building out** from the piers or abutments without staging is a system suitable for the erection of metal arches and cantilevers.

The feasibility of building out large metal arches from the springings without any temporary intermediate support till they meet in the centre of the span is one of the chief advantages of the metal arch type of bridge when built over a deep narrow valley or across a broad and deep river flowing with a high velocity. The steel tubes of the upper and lower members of each rib of the St. Louis bridge arches were built out in 12 feet lengths from the springings with their intermediate bracing for a quarter of the span on each side, being supported like a bracket by their attachments to the pier. The other quarter portions up to the centre of the span were gradually extended by supporting the projecting lengths by a series of iron bars passing over a temporary timber tower erected on the pier and fastened to the corresponding projecting length of rib on the opposite side, the whole forming a temporary balanced double cantilever with a similar arm on each side of the pier. At the abutments, the tie-bars passing over the tower were secured to the shore viaduct behind, there being no corresponding length of cantilever on the shore side as in the case of a pier. The changes in length of the long coupled bars supporting the projecting ribs due to variation in temperature were adjusted by raising or lowering the supports on top of the tower, these supports being moved slightly up or down by regulating

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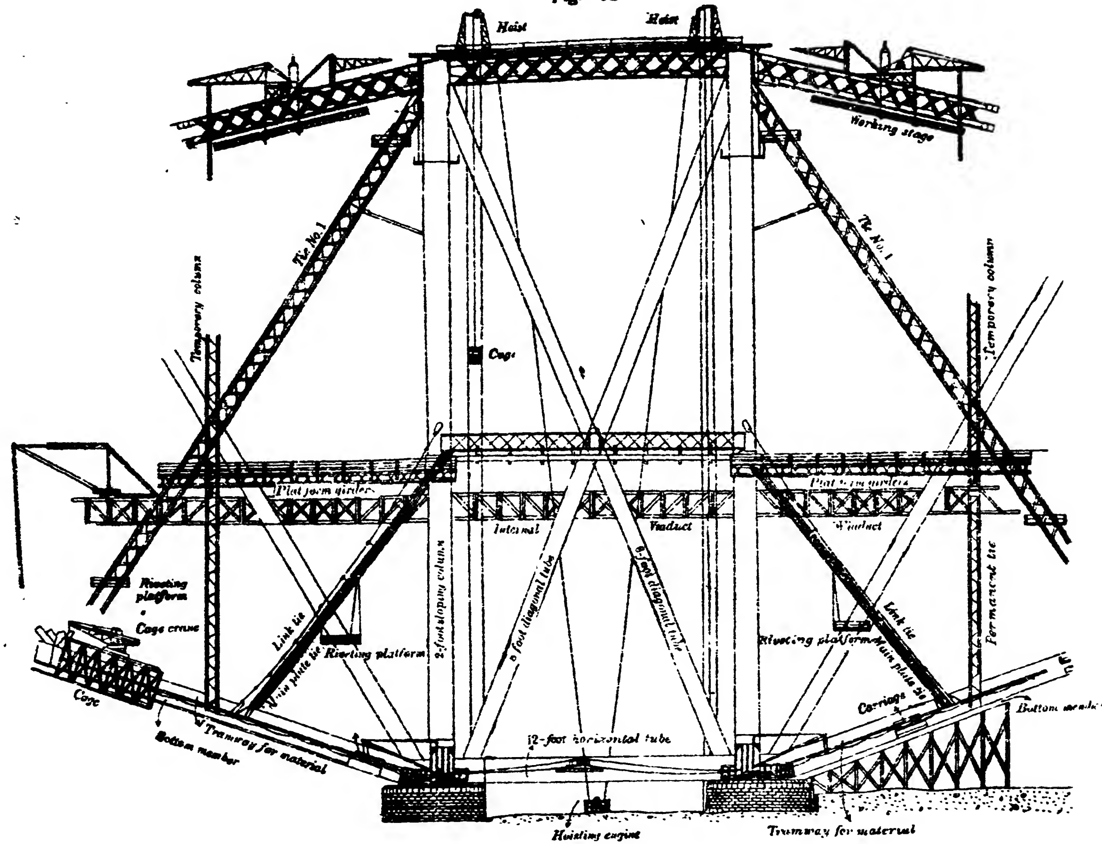
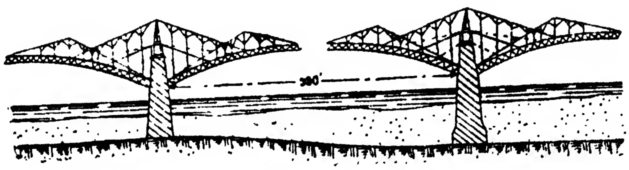
Fig. 132.



P.S.U.P.(R)-4, S.R.S.-1947-1250.

the pressure of hydraulic plungers on which they rested. See Fig.* 131.

Fig 131



Each half span of the Niagara Falls and Clifton Arch was built out from the abutments by a very similar method.

Fig.133.

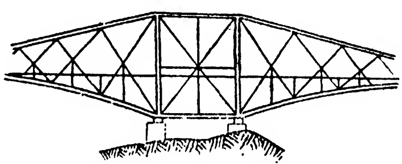
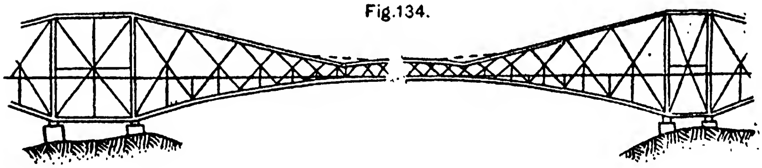


Fig.134.



Building out from piers on each side is specially suitable for the erection of symmetrical double cantilever bridges such as the Forth and Niagara bridges already described because the cantilever portions are subjected in the process of erection to exactly the same strains as those which they are designed to bear when completed. It is only the relatively short central girder which does not share this advantage as it has to be built out in halves projecting from the cantilver and tied back temporarily by strong ties connecting its upper flange with that of the cantilever at their junction; these ties are removed when the central girder is connected in the middle and rests as an independent girder on bearings in the end pillars of the cantilevers. The erection of the Forth bridge by building out symmetrically and simultaneously from the piers

*Figs. 133, 134 from "Civil Engineering" by Vernon Harcourt. By permission of publishers.

is illustrated at three different stages in Figs.* 132 to 134. The temporary ties connecting the upper flanges of the central girder and cantilevers during erection should be specially noted in Fig. 134 where they are shown by dotted lines. Fig. 132 shows the lifting arrangements by which pieces were raised from boats on the sea below the structure and placed in position and the working stages from which the pieces were put together and riveted.

228. **Testing girders after erection.**—When a large girder bridge has been completed, the girders are usually tested before being brought into use for deflection and oscillation under the heaviest moving load to which they are likely to be subjected. If the bridge is on a railway, a heavy train consisting of two locomotives and the longest line of goods waggons ever likely to be used are first brought very slowly on the bridge and allowed to stand on it while readings of deflection are taken in the centre of the span. The train is run off and the permanent set noted. The train is then run over the bridge at the maximum permissible speed and the deflection and oscillation of the lower boom during transit are noted. The readings are taken automatically on two pieces of paper fixed horizontally and vertically on boards secured to an independent staging erected from the river bed up to the level of the lower boom but not touching the iron work anywhere. Pencils working against springs are clamped to the lower boom pressing firmly against the test papers—one vertical, the other horizontal—and the movements of the girder are automatically recorded by the pencils. The vertical reading represents the deflection and the horizontal the oscillation.

A CHAPTER ON STEEL BRIDGES

By L. H. Swain, B.Sc., A.M.I.C.E., Bridge Engineer, Oudh and Rohilkhand Railway, Lucknow

PRELIMINARY. Section 1

1. **Evolution.**—The science of bridge-building affords an interesting example of evolution, and a study of steel bridges may be advantageously prefaced by a consideration of the simple beam and the forces and stresses which act in it. There is no doubt that the log of timber hewn down and thrown across a gap was the earliest example of bridge-building employed by mankind. Subsequently when a more regular beam was required, the logs or tree trunks were "squared" and we come to the form of the simple rectangular beam which even survives today in those countries where good straight trees are plentiful. In America and Australia the timber viaduct has been and is still largely employed in pioneer construction. The timber beams employed in this work are of standardized dimensions, usually 18 inches by 9 inches, 20 inches by 10 inches, 24 inches by 12 inches, etc. The larger dimension indicates in every case the way in which the timber is to be placed, since the strength of a beam of given area is proportional to its depth.

2. **Theory of Bending.**—The simple theory of bending has formed the basis of evolution in design. Consider the forces which act across the section of a beam, *vide* Fig. 1.

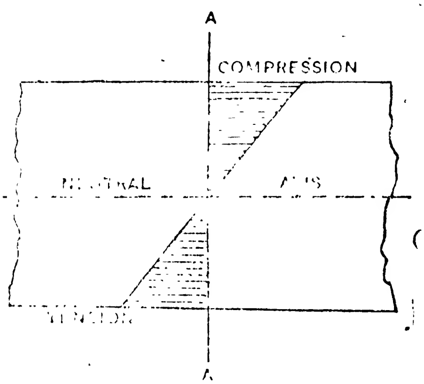


Fig. 1.

In any beam there is some intermediate line called the neutral axis about which the beam bends and the stress in the material of the beam increases on either side of this line as we recede from it and approach the top or bottom of the section. The maximum stresses are reached at outer top and bottom fibres. The stresses on the two sides of the neutral axis are of opposite sign. In a simply supported beam the top is in compression and the bottom side in tension and evidently the criterion of strength of the beam is the maximum or skin stress induced in compression or tension in the top or bottom fibres respectively.

According to the size of the scantling and the maximum permissible stress allowed either in tension or compression, we get a certain resistance to bending of the beam which is called the Moment of Resistance.

3. Moment of Resistance.—The moment of resistance of a beam is the measure of its strength.

In the case of a simple rectangle, the moment of resistance is calculated by the well-known formula—

$$M_r = f \times \frac{b\,d^2}{6},$$

where f = maximum permissible skin stress
b = breadth of the beam
d = depth of the beam

This formula is of course a special case of the general formula—

$$M = f \times \frac{I}{y}$$

where M = bending moment
f = the stress in any fibre
I = Moment of Inertia of the beam section about the neutral axis
y = the distance from the neutral axis of the fibre corresponding to the stress f.

Returning to the simple beam where

$$M_r = f \times \frac{b\,d^2}{6},$$

it may be observed that the moment of resistance is proportional to the area of the beam section $A = b\,d$, multiplied by the depth and may be writtten

$$M_r = f \times \frac{A}{6} \times d.$$

Hence it is apparent that of the total area of the section the equivalent of only one-sixth part (Fig. 2) is fully occupied at top or bottom to resist bending. The rest of the timber so far as this purpose goes is wasted. It is therefore evident that the rectangular beam is not an economical one. Moreover it would appear at first sight that one could cut away or do without the middle two-thirds of the section. This however omits the consideration of another feature in the mechanics of a beam, namely the shear action, for which allowance must be made.

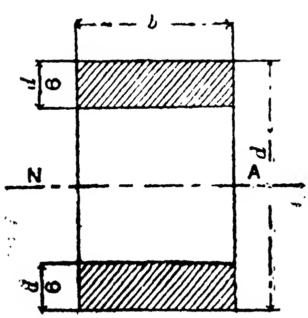


Fig. 2.

4. **Shear.**—The shear forces in a beam are the direct effect of the vertical external forces acting on it which include the loads on the beam acting downwards and the reactions of the supports acting upwards, *vide* Fig. 3. The nett shear F on any section A-A is the summation of the loads and reaction to one side of it having due regard to the sense or sign of the forces. The nett shears F_1 and F_2 on the two sides of the section are equal and opposite in sign.

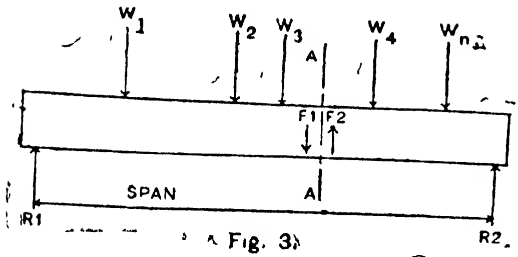


Fig. 3.

This is evident from considerations of equilibrium and constitutes the state of stress known as shear. The total shear force F acting on a vertical section of a rectangular beam is distributed over that section in the form of a parabola as shewn in Fig. 4. The horizontal ordinate at any point on the vertical section gives the intensity of shear stress at that point. The maximum shear stress occurs at the neutral axis and is zero at the top and bottom of the section. Further it may be observed that the bulk of the shear is taken on the middle two-thirds which part formerly appeared to be wasted. Hence the middle portion of the beam is doing some work.

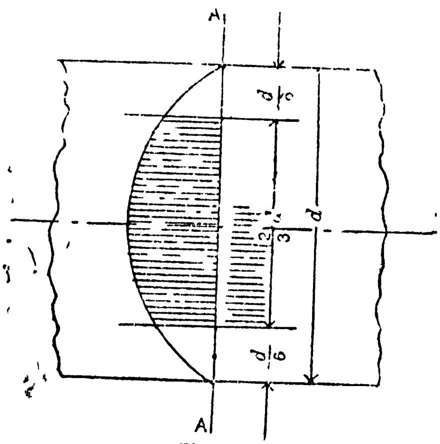


Fig. 4.

It will be found, however, in the case of a rectangular section that the maximum intensity of shear is not up to the standard permissible. There is therefore a surplus of material which can be dispensed with. In practice the requirements are met by leaving a vertical strip of section called the web and the economical girder takes

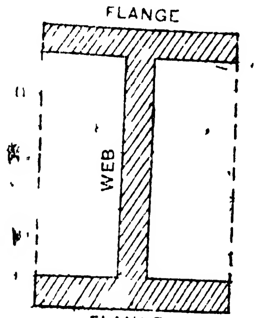


Fig. 5.

the form shown in Fig 5. This is the general form of steel beams and plate girders.

5. **"Principal" stresses in a Beam.**—The combined effect of bending and shear action at any point in a beam can be represented by two principal stresses (*vide* Ellipse of stress) at right angles and it is highly instructive to draw the lines of principal stress in a loaded beam

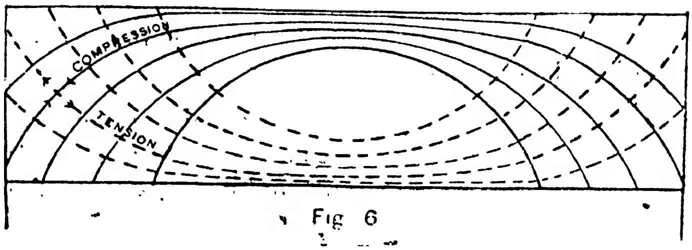


Fig 6

as shewn in Fig. 6. The full lines are compression lines and the dotted lines are tension lines. A study of this diagram will give an immediate conception of the principal forces in a girder, and it will be apparent that a beam considered fundamentally is a combination of an arch and a suspension bridge. This has a practical application in the construction of reinforced concrete beams. It also indicates what the lines of evolution are in the case of a lattice or open web type of girder.

THE ECONOMICAL SPAN. Section 2

6. **Controlling Features.**—Assuming that the location and length of a bridge have been determined by general considerations of alignment, river course, flow of water, clear waterway, etc., the length of a unit span has to be decided. The length of unit span adopted will depend on the nature of the river bed and the height of the piers and the depth of foundation required. In special cases in navigable rivers the span may be determined by the necessity of providing a certain width of clear waterway and a certain height for vessels to pass under. The ordinary procedure however where these considerations do not apply is to determine the length of span on economical grounds by balancing the cost of the foundations against the cost of the superstructure.

7. **The Economical Span.**—The economical span is determined as follows. Assuming that the width of the required bridge is fixed by the necessary provisions for traffic including number of railway tracks, width of roadway, footways, etc., as the case may be, then the cost of the

flooring system may be assumed as a fixed amount. It includes cross girders, longitudinal bearers, track timbers, troughing, ballast, paving, decking, etc., which represents a certain cost per foot run independent of the length of a span. The cost of the main girders for a given length of bridge, however, will increase proportionately to the length of the spans adopted. The weight of a girder to carry a fixed load per foot run is proportional to the square of the span, since the maximum bending moment $M = \frac{W \times l^2}{8}$, where W = weight per foot run and l=span in feet.

If n = number of spans in a bridge of length$=L$;

P = cost of one pier,

G = cost of one pair of main girders

$= k\, l^2$ say, where k is a constant;

then the total cost of main girders and piers

$= n\,(P + kl^2)$ which must be a minimum.

But $n = \frac{L}{l}$,

therefore total cost

$$C = \frac{L}{l}(P + kl^2). \qquad \text{(1)}$$

Differentiating this with regard to l,

$$\frac{dC}{dl} = L\left(\frac{-P}{l^2} + k\right).$$

For this to be a minimum, $\frac{dC}{dl} = 0$

hence $\frac{P}{l^2} = k$

therefore $P = kl^2 = G$.

Hence the cost of one pier should be equal to the cost of one pair of main girders.

It may be noted that the above calculation assumes that the cost of one pier will be fixed and independent of the span. This is not strictly true in a wide range of spans, although the depth of the foundation will be fixed and there is a certain minimum size of pier or well that can be adopted, and the assumption is sufficiently true for the range of span envolved in the consideration of any particular case. The proposition may be summarized in a few words. For a given total length of bridge, the cost of the piers increases as the number of spans, and the cost of the main girders decreases as the number of spans, and the greatest economy is reached when the total cost of the main girders and the piers is equally divided. This fixes the length of span.

8. **Standard Spans**:—In the construction of railways in India the following standard lengths of span are used:—

Plate girders		*Lattice girders*	
10	ft.	100	ft.
15	"	150	"
20	"	200	"
30	"	250	"
40	"	300	"
60	"	350	"
80	"		

The same standard will apply to the spans of steel road bridges where put in as in general they are constructed subsequently to the railway bridges and the piers are placed conformably.

9. **Clear Span.**—As a rule when a certain span is nominated, *e.g.*, at 100 ft. span it means clear span as distinct from the effective span.

10. **Effective Span.**—The effective span is fixed by the position of the pier reaction and is the basis for design and calculation of the girders.

DESIGN AND SPECIFICATION. Section 3

11. **External Forces.**—The dimensions of a girder are fixed by a consideration of the maximum bending moment and shear produced on any part by the following external forces which may combine at any one time to produce stress in a member:—

(1) Fixed load of structure.

(2) Moving load on the bridge.

(3) Wind and other forces causing lateral deflection and oscillation.

(4) Traction and braking forces.

(5) Forces due to horizontal curvature of track.

(6) Forces due to change of temperature.

12. **Fixed Load.**—The fixed or dead load of a structure is the total weight of the material in one span supported between the bearings. It is determined by computation. The magnitude of the fixed load is not known accurately until the bridge is designed, so for the purpose of preliminary calculations it is necessary to assume some figure for the fixed load obtained either from a formula giving the weight of type spans to carry a certain load for different span lengths, or by comparison with

similar structures. Due account has to be taken of any variation from previous types, and by experience it is generally possible to make a fairly close preliminary estimate of what the dead load will be.

13. **Moving Load**—The moving load comprises the live load effect of the traffic on the bridge. It is fixed according to certain standards according to the class of traffic for which provision is made. The standard employed will be that standard of loading which covers the heaviest traffic which the bridge may be expected to carry in its lifetime. In the case of railway bridges, the heaviest train loads are covered by tables of loading which specify for all spans the unit load in tons per foot run of track which if uniformly distributed over the length of bridge under consideration will give the worst effect of any train load which the standard is intended to cover.

A copy of the Government of India Tables for ("B"+25%) Standard Loads for Railway bridges of various spans is appended to the Manual.

14. **Impact Allowance to Moving Load.**—It is important to observe that allowance of moving load must include besides the nett effect of the load according to standard, an additional load to cover the dynamic effect of movement which causes the moving load to produce a greater deflection and greater stresses in the members whilst passing at speed over the bridge than it would do as a mere static or stationary load. The allowance for impact is added to the nett live load, and the figure thus obtained is used as a static load to determine the total effect on the members.

The increment for Impact on Railway bridges to be allowed on the moving train load (Mt) is to be calculated by the formula.

$$\text{Impact allowance } \frac{300}{300+L}(Mt) \qquad \text{(Railways)}$$

where L is the length in feet of that portion of the span which the train has had to traverse to reach the position which gives the maximum stress in the member under consideration.

In the case of Road bridges the impact allowance may be taken as only half the allowance required for Railway bridges, namely,

$$\text{Impact allowance } \tfrac{1}{2} \times \frac{300}{300+L}(Mt) \qquad \text{(Roads)}$$

15. **Wind loads and racking forces**—One of the most essential features of modern design is the provision of adequate and rigid system of bracing to resist transverse forces which come into play on a bridge due (1) to the horizontal force of the wind which, acting

on the exposed area of the bridge and also on the area of the train whilst on the bridge, causes the span to act as a girder in a horizontal plane, and (2) the oscillation due to unbalanced forces in the locomotive necessitating the provision stronger wind bracing in the horizontal plane than is required for wind effects alone and also rigid bracing at intervals in a verticl plane connecting the two main girders together.

16. **Traction and braking forces**—The traction and braking forces are of considerable importance in high viaducts as affecting the stability of the piers, which may need special consideration in some cases. The tractive force or breaking effort puts a longitudinal stress in the bridge parallel to the girders. It acts first on the flooring system and directly affects the stresses in the wind bracings, and is thence transmitted to the booms of the girders and from them to the bearings and to the piers.

17. **Forces due to horizontal curvature of the track**—The external forces included under this head are those which occur in the case of bridges where the track is on a curve and include both the horizontal effect of centrifugal force on the moving load and the eccentricity of loading on the main girders. The latter effect cf curvature causes an unequal distribution of load on the two girders and in some cases the excess on one girder may be considerable.

18. **Temperature stresses**—Stresses due to change of temperature are as far as possible eliminated by the provision of expansion joints or bearings and normally they need not be considered in the cases where expansion is provided for as in girder bridges. They are of most importance in the case of arch bridges of the two pin or rigid type. It may be remarked however that where one end of a span is made movable to permit of expansion movements, the practical features of design introduced must be attended to, and this applies particularly to spans of 100 feet and over which require one end of a span to be placed on expansion rockers.

19. **Structural steel British standard specification**—The steel employed in the manufacture of bridges must be up to a certain standard of quality which has been specified in the British Standard Specification as follows:—

The steel shall be made by the Open Hearth Process (Acid or Basic) and must not shew on analysis more than ·06 per cent. of sulphur or phosphorus. The tensile breaking strength of the steel sections must not be less than 28 tons per square inch nor exceed 33 tons per square inch with an elongation of not less than 20% on an 8-inch test length. Rivet steel must have an ultimate tensile strength of not less than 25 tons per square inch and not exceeding 30 tons per square inch with an elongation of not less than 25% on an 8-inch test length.

20. **Working stress for steel bridges**—Based on the above standard of material used, the following unit working stresses may be permitted in the structure.

The figures given are the maximum permissible stresses and are in accordance with the latest British standard specification for Girder Bridges No. 153—Parts 3, 4 and 5—1923.

	Steel *Tons per sq. inch.*
(1) Tension, axial, on net section ...	8
(2) (*a*)—Compression, on gross section of compression boom of plate girders	$8\left(1-\cdot0075\frac{L}{W}\right)$
L is the greatest unsupported length and W is the width of the boom, provided that the gross area of the compression flange shall not be less than the tension flange.	
(*b*)—Compression, on gross section of compression members of truss and lattice girders with rivetted connections	$8\left(1-\cdot0033\frac{L}{r}\right)$
(*c*)—Compression, on gross section of compression members of truss and lattice girders with pin connections	$8\left(1-\cdot005\frac{L}{r}\right)$
Subject to a maximum stress under (*b*) and (*c*) in all cases of	6·8
L is the greatest unsupported length of the member and *r* is the least radius of gyration.	
(3) Shear, on gross section of web plates	5
Shear on shop rivets and tight-fitting turned bolts	6

Shear on hand-driven field rivets and black bolts 5

(4) Bearing, on shop rivets and tight-fitting turned bolts 12

Bearing, on hand-driven field rivets and black bolts 10

In calculating the intensity of stress due to any combination of loading, it is permissible to ignore the effect of wind pressure on the members of main girders of bridges providing such effect does not exceed 25% of the total working stress (excluding wind). This relaxation is based on the assumption that during a gale of wind, the train will move at such low speeds that the effect of impact will be very small.

21. **Bending moment and Shear Action**—Before the scantlings of a girder can be fixed it is necessary to know the maximnm value of the bending moment effect and the shear action at all points along the span.

The bending moment and shear diagrams for simple beams are familiar to the students of Applied Mechanics and he knows the shape of the diagrams in simple cases. The diagram, *Fig.* 7. shews the variation of bending moment on a uniformly loaded beam and is represented by the segment of a parabola. The shear diagram for the same loading is a straight line cutting the base line with zero value at the centre of the span. It is apparent that the shear at any point is a measure of the rate of change of the bending moment and is in fact proportional to the differential coefficient of the equation to the bending moment curve. This is true for any system of static loading.

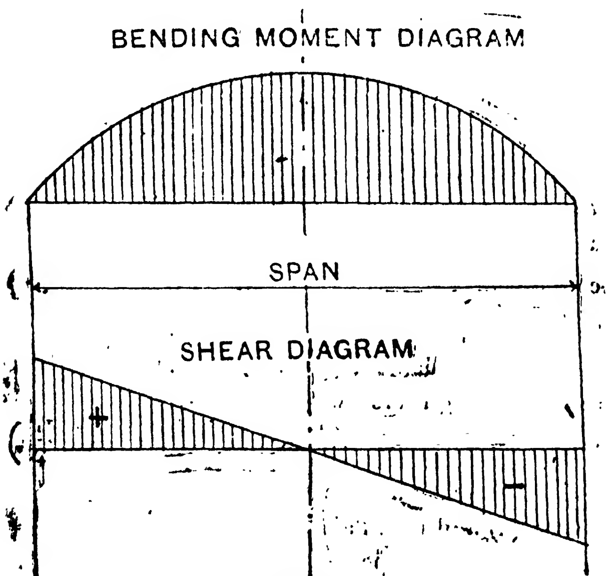


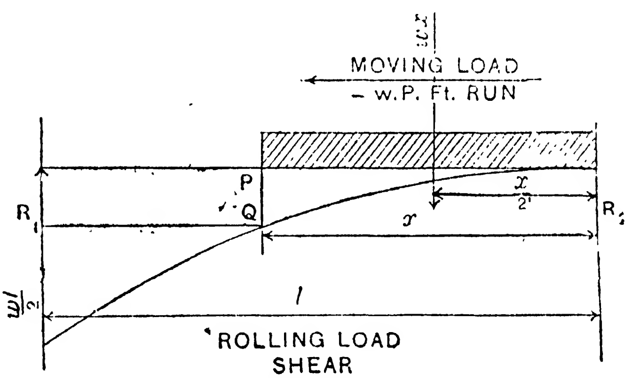
Fig. 7.

The particular case shewn in *Fig.* 7 gives a general conception of what the bending moment and shear variation will be on any bridge. It will be observed that the booms which will resist the bending moment will have to be strongest at the centre of the span, and the web system which resists the shear will have to be strongest at the ends

22. **Rolling Loads**—In the case of a uniformly distributed rolling load coming on to a span the value of the bending moment and shear effects vary according to the position of the load.

In the case of the bending moment it is evident that the maximum effect for all points along the span occurs when the span is fully covered by the load, and the diagram of maximum bending moment is a parabola as before *vide Fig.* 7. The maximum ordinate $= \frac{w\,l^2}{8}$ where w is the unit load per foot run and l = the length of the span.

In the case of the shear under a rolling load, however, the maximum value of the shear at any point along the span occurs when the front of the moving load has just reached that point in traversing from the further abutment. Consider the maximum shear at point P. *Fig.* 8.



ROLLING LOAD
SHEAR

Fig. 8.

If x distance from the further abutment, shear at P. is equal to the reaction. R_1.

By moments $R_1 \times l = Wx \times \frac{x}{2}$

$$R_1 = \frac{wx^2}{2l}$$

i.e. $PQ = R_1 = \frac{wx}{}$ .... (1)

Hence the equation to the curve of maxmum shear is also a parabola and the maximum value is when

$$x = 1$$

$$i.e.,\ S\ max. = \frac{w\ l.}{2}$$

23. Influence Lines—It is of interest to note a convenient method of determining the variation of shear due to rolling loads by means of influence lines.

A simple case is shewn in fig. 9. AC and BD are set off to a scale equal to w the unit rolling load per ft. run. Join CB and AD and draw a vertical through P. Then the area of the triangle PQB gives the shear due to reaction R_1 and the area of PRA gives the shear due to reaction R_2 if the load comes on the other way.

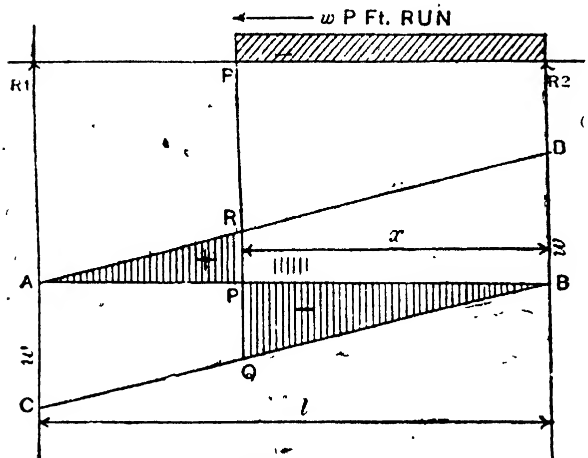


Fig. 9

The area of

$$PQB = \frac{PB \times PQ}{2}$$

$$= \frac{x}{2} \times \frac{w\ x}{l}$$

$$= \frac{w\ x^2}{2\ l} = R_1$$

Similarly area of PRA$=R_2$ when the load comes on from left to right.

PLATE GIRDERS. Section 4,

24. Use and Limitations.—Plate Girders are employed for bridges up to 80 feet span where heavy loads have to be supported. They are the standard type of girder for railway underbridges up to that span and their relatively simple construction and durability in services render them of particular utility. For spans of over 80 feet economy in material demands the introduction of the open web or lattice girder which will be discussed subsequently. The evolution of the plate girder has already been described. It is now intended to deal with the design of a plate girder and the determination of its scantlings for a specified standard of loading.

25. **Standard Types.**—As plate girders are largely employed for railway deck spans it will be convenient to deal with the features of standard girders employed on railways in India designed up to the 1908 Government of India standard of loading for railway under-bridges.

26. **Depth over angles**—The depth of a plate girder should be proportional to its span and the economical depth is generally one-tenth of the span.

The following table gives the "depth ratio" for standard plate girders:—

I. S. R. STANDARD PLATE GIRDERS. (5′-6″ GAUGES).

1908 Loading

Clear Span S Ft.	Depth Over Angles D Ft.		Ratio $\frac{S}{D}$
10	Rolled steel beams	1′-6″	6·67
12	Do.	1′-8″	7·2
15	Do.	2′-0″	7·5
20	Do.	2′-0″	10·0
30	Built girders	3′-3″	9·23
40	Do.	4′-0″	10·
60	Do.	5′-9″	10·67
80	Do.	7′-3″	11·02

27. **Dead Weight.**—The following gives the nett weight of steel work in standard spans:—

I. S. R. STANDARD DECK SPANS (5′-6″ GAUGE).

1908 Loading.

Clear Span Ft.	Nett Weight Tons.
10	1·36
12	1·85
15	2·28
20	3·99
30	7·67
40	11·96
60	24·27
80	44·84

For spans of 40 feet clear and over the following formula gives very closely the weight of 1908 standard spans for single track.

$$D = \cdot 007\, l^2 \qquad \text{FORMULA.}$$

where D weight in tons

l clear span in feet.

28. **Effective Span of Plate Girders.**—The effective span depends on the length of the bearing plate. The following table gives the particulars for standard spans and the effective span is determined by assuming that the reaction of the support occurs at a point one-third of the length of the bearing plate from the edge of the support. This is based on the assumption that when a span is loaded the deflection causes the weight to be distributed on the bearing in proportion to a triangle of which the centre of gravity is one-third from the face :—

I. S. R, STANDARD SPANS

Clear Span	*Length of Bearing Plate*	*Over All Length.*	*Effective Span.*
Ft.			
10	1′-4″	12′.8″	11′-1″
15	1′-6″	18′-0″	16′-0″
20	1′-6″	23′-0″	21′-0″
30	1′-9″	33′-6″	31′-2″
40	2′-0″	44′-0″	41′-4″
60	2′-6″	65′-0″	61′-8″
80	3′-0″	86′-0″	82′-0″

29. **Thickness of Web.**—The first scantling to fix in a plate girder is the thickness of the web plates which is determined by two considerations. The theoretical consideration is the maximum shear stress, and the practical consideration is the minimum pitch of rivets to be employed in connecting the boom angles to the web at the end of the girder.

The maximum shear is determined by considration of the total dead and live load and the consequent maximum reaction at one support.

Let D = dead weight of span including weight of decking and track which must be added to the nett weight of steel work

then Dead load Reaction = $\frac{D}{4}$ since the span is supported at four points.

The live load is determined from the standards given in table 3, p. 38. Referring to the latter which covers a range of spans from 5 to 500 feet it will be observed that against each span is given the load per foot for both 5′-6″ and metre gauge loadings. Call this w then referring to para. 22 the maximum end shear is equal to $\frac{w \times l}{2}$ which is divided on two girders. At this sta ge the impact factor must be introduced and we get the maximum live load reaction at each bearing including impact

$$M = \frac{wl}{4}(1 + i) \text{ where } i = \frac{300}{300+l}$$

If d = depth of girder

t = thickness of web

and fs = shear stress

Total end shear reaction on one girder

$$S = \frac{D}{4} + \frac{wl}{4}(1+i) = d \times t \times fs$$

If fs is fixed, then t is determined.

In practice it will be found in modern deep girders that t determined in this way will be extremely small and in all cases it is necessary to check the thickness thus determined by calculating the bearing stress in the rivet holes.

In the above calculation it is assumed that the shear is uniformly distributed on the vertical section of web. This is not strictly true because, as was pointed out in para. 4, the distribution of shear on a vertical section of a beam is parabolic. The assumtion of uniformity is an approximation which is commonly adopted in plate girder design for simplicity. For the purpose of determining the rivet pitches at top and bottom the assumption is on the safe side though it is found to be rather wide of the mark where investigations of old girders are under consideration. For the purpose of design, however; we can assume that the state of shear induced in the web is the direct effect of the horizontal pull and thrust on the rivets connecting to the top and bottom flanges, *vide* fig. 10, Consider a vertical strip of web of width equal to one rivet pitch p.

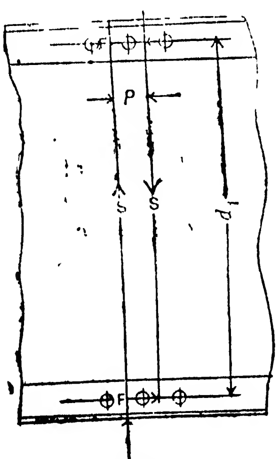


Fig. 10.

If d_1= distance between the upper and lower lines of rivets and F= horizontal force on one rivet, the vertical shear couple is balanced by the horizontal couple and we have

$$S \times p = F \times d_1$$

If fB=bearing stress in the rivet holes of area a then F $=f$B$\times a$.
therefore S$\times p = f$B $\times a \times d_1$.

"a" is fixed by the size of the rivet and the thickness of plate and it is necessary to work out what the value of fB is, which should not exceed 12 tons per square inch. The pitch "p" cannot in practice be made much less than 3" so that this is a criterion which will determine "a" and hence the value of "t".

In practice "t" should never be less than 3/8" and generally for economy will not exceed ½" if d_1 is made sufficient.

Having fixed the end thickness of web the thickness throughout the girder will depend on practical considerations. It is common to maintain the same thickness of web throughout. The thickness of web requires to be known before the flange section can be designed.

30. **Flanges.**—The flanges of plate girders are generally composed of two main angles which make the connection to the web, and a certain number of flange plates.

For the main angles of built up girders it is common to employ ar large size section 6 inches by 6 inches by a sui table thickness, fo girders from 30 feet to 60 feet span. For larger spans the main angles may be 8 inches by 8 inches.

The advantages of a wide section of angle are it's stiffness and the fact that double rivetting and a generally simpler construction of girder is secured. The main angles usually extend from end to end of the girder and besides forming an integral part of the boom to resist the horizontal forces of bending, they also provide local rigidity to the flanges at points of concentrated loading, *e.g.*, where the sleepers are placed in the case of deck spans.

The scantling of main angles and the size of flange plates required will depend on the moment of resistance required which must be equal to or in excess of the maximum being moment to which the girder will be subjected by the external forces.

31. **Maximum Bending Moment.**—The maximum bending moment will be determined by the dead load bending moment and that

due to the live and impact load. The live load is determined by reference to Table I which specifies the total load for any span and the load per ft. run which are synonymous (*i.e.* $W=wl$).

We now consider the total load on one girder, and the maximum bending moment is then

$$B.M. = \left\{\frac{D}{2} + \frac{W}{2}(1 + i)\right\}\frac{l}{8} \quad \text{where } i = \frac{800}{800+l}$$

In order to equate the above bending moment to the moment of resistance of the girder it is necessary to determine the effective depth of the girder and decide what portion of the total section may be considered as resisting bending moment.

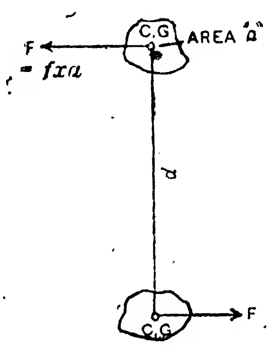


Fig. 11

It was shewn in para. 8 that the moment of resistance of a rectangular beam could be expressed as

$$MR = f \times \frac{A}{6} \times d$$

and graphically this is represented by a couple Fig. 11.

In the case of a plate girder, we take the effective area of the flange "a" (equivalent to $\frac{A}{6}$ in the above equation).

32. **Nett Flange Section.**—The nett section of a plate girder flange is to be calculated on :—

(1) The nett section of the two main angles

(2) *plus* the nett section of the flange plates

(3) *plus* one-eighth of the full section of the web plate.

By the nett section is meant the area after deducting rivet holes.

Now if we assume a uniform stress of 8 tons per square inch over this area and calculate the position of its centre of gravity, we fix the effective depth of the girder and the moment of resistance is equal to $MR=a\times 8\times d$ which must be equal to or greater than the maximum bending moment.

As a first approximation it can be assumed that the centre of gravity of the boom section will be about one inch below the root of the main angles. Hence "a" is fixed.

We know items (1) and (3) in the above nett area, hence we determine item (2).

In actual practice a certain amount of trial and error is involved because the problem is effected by practical considerations of standard sizes of flange plates and the arrangement of rivetting and the number of rivet holes which come in line, which must be deducted.

A study of the standard type spans will shew that in the case of a 30 feet span no flange plates are used as the angles are strong enough without. In the 80 ft. type two—$19'' \times \frac{1}{2}''$ plates are used in conjunction with two $8'' \times 8'' \times \frac{3}{4}''$ angles.

In ordinary plate girder design the bottom boom only is calculated allowing for the deduction of rivet holes and the top boom is made the same as the bottom boom and is relatively stronger since well fitted rivet holes are not deducted in compression.

33. **Sizes of Rivets.**—The sizes of steel rivets used in structural plate work depend on the thickness of the plates rivetted together.

The following is the practical rule :—

Thickness of individual plates joined	Diameter of rivets.
$\frac{1}{4}''$	$\frac{5}{8}''$
$\frac{3}{8}''$	$\frac{3}{4}''$
$\frac{1}{2}''$ to $\frac{3}{4}''$	$\frac{7}{8}''$
$\frac{3}{4}''$ & over	$1''$

In certain cases where special shear strength is required on in cases of an excessive length of rivet where cooling stresses may be excessive larger sizes of rivets may be used.

The above diameters refer to the size of the holes. The actual rivet before driving will be $\frac{1''}{32}$ less in diameter but it is supposed to fill the hole after driving hot.

34. **60 Ft. Standard Deck Span.**—It will be convenient at this stage to consider further details of design by reference to a standard 60 ft. girder.

The calculations applying to this are given on a stress sheet accompanying the type drawing in the Appendix (*see* Plate no. 8).

35. **Curtailment of Flange Plates.**—Having fixed the number of flanges plates it is usual to determine how these can be curtailed where the B. M. is less towards the ends. This is best done graphically. The

individual moments of resistance of the various components of the flange at full section are worked out and the curtailment determined in the manner shewn on the stress sheet, allowing about 18" to 2 ft. overlap. In the case of the top flange in the 60 ft. design, the first flange plate is carried through right to the end. This is not strictly necessary but provides a more uniform and better looking top to the girder.

Whilst dealing with the flange plates and their curtailment it is advisable to consider the position of the boom joints if required and the provision of suitable cover plates.

36. **Joints.**—The joints in girders are as a rule determined by purely practical considerations. There are two main considerations.

1. Transportation.

2. Maximum length of rolled sections procurable. The first has two features bulk and weight and both may be limited. In special cases considerable feats of transportation have been done by rail but for ordinary purposes the maximum length of steel girder pieces shipped to India should not exceed much over 40 feet. If portions longer than this are shipped they are liable to get damaged and sometimes cranked with rough handling. Many engineers specify that a built girder should never be laid on its side, as obviously it has very little lateral resistance this way and many become permanently set even under its own weight.

Item 2 conforms to item 1. As a rule flange plates or angles can be obtained if necessary considerably over 40 feet, so that first factor limits.

Web plates, however, are limited in total area which rarely exceeds 200 square feet. Special plates can be rolled if necessary but may be more expensive.

The 60 feet girder is built in two halves so that web and boom joint up at the same place.

The design of cover plates for the joints in girders requires practical experience. The strength of the joint should not be less than the strength of the nett scantling The number of rivets must be in excess of requirements of nett shear in the overlapping portions of the joint.

37. **Rivetting.**—To determine the number of rivets in the lap, the tensile resistance of the plate or angle as the case may be equated to the shear resistance of the number of rivets, and 20 per cent. in the case of field joints added to the number found to allow for imperfect rivetting. This rule is being modified and it is now considered that 10 per cent. excess for field rivetting is sufficient.

38. Cover plates.—A note is here required in reference to thickness of cover plates. These are usually made a little heavier than the metal which they cover.

In the stress sheet for the 60 ft. girder is shewn a mid section through the cover plates and on account of the rather complex position of the covers the strength of the joint is most easily and accurately checked by determining the moment of inertia of the section and calculating the skin stress by the formula.

$$B.\ M. = \frac{f \times I}{y}$$

The stress is under 8 tons per square inch the permissible, hence the joint is good.

Web covers are provided. They are double spliced. There is no special calculation applicable as the rivetting will generally be surplus to shear requirements.

39. Stiffeners.—The next thing is stiffeners. These are commonly fixed at 3'-6" centres as a practical rule. They are an essential part of the girder in stiffening it up and preventing twisting. These are always required at points of special load concentration. They are useful in deck span for assisting in the connection of sway bracing.

40. Bearings.—There is now one other special point in the main girder ; the bearing. The student will note this is stiffened up by extra side plates.

Assuming that the reaction of the bearing is directly taken on the boom angles and thence transmitted to the web, by the provision of the side plates, the total number of rivets in single shear is 40 or rather 10 rivets in quadruple shear 120 tons as against a maximum reaction of 90 tons.

41. Bracings.—In the case of plate girder deck spans two kinds of bracings are required known as wind and sway bracing. These are shewn in plate no. 2.

Wind bracings should form a triangulated system in a horizontal plane and are connected to the top booms of the two girders.

Sway bracing usually occurs in a vertical plane at panel points, or at intervals in the span in a plate girder.

The end sway brace frames are usually made specially heavy to brace up the girders and to resist the vertical and horizontal shear reactions.

In the new 60 feet design it will be noticed that the horizontal wind bracing is made of 6″×6″×7/16″ angles throughout.

This is obviously not required to resist wind shear and in the earlier designs of 60 feet spans the bracings were reduced towards the middle to conform to wind loads. It has been found however, that the prevention of oscillation requires a rigid and heavy bracing throughout.

42. **Standardization of Details**—There is a general tendency in modern structural design towards standardization of details. This applies to gussets and wind bracing connections and for any connections throughout a span where the maximum requirements can be met by a standard detail or part without sacrifice in economy of weight. A considerable economy in manufacturing processes is thereby effected and it is possible by interchangeability to greatly facilitate erection at site.

LATTICE GIRDER BRIDGES. Section 5.

43. **Lower Limit of Span.**—When the span of a girder exceeds 80 feet the ordinary plate girder ceases to be economical. In very special circumstances plate girders have been made of 100 feet span and over, but for spans in this region, in order to preserve the proportion of depth to length, the amount of material required in the web of the girder is excessive and the advantages of the plate girder no longer hold. Accordingly for spans over 80 feet the open web or lattice type of girder is used.

There is an exception to the above rule in favour of lattice girders for light foot-bridges which are generally of lattice type for all spans of 40 feet and over.

44. **Features of Lattice Girders.**—The transition from the relatively simple plate girder to the open web type introduces many new features and requires the application of the principles of statics of triangulated structures. The girder has now become a cadre or skeleton structure the members of which are so placed and proportioned as to meet the primary needs and forces which occur in a beam. These forces are now concentrated in simple axial loads in the members which are placed in the most suitable way to resist them.

The evolution of the lattice girder has resulted in many different types, of which, the most important features may be classified under two main heads of differentiation.

These heads are :—

(1) Continuous girders over two or more spans and simply supported unit spans.

(2) Girders with different systems of web bracing and panel formation.

The question of continuous girders *versus* simply supported unit spans is of primary importance in special cases and from the point of view of economic, since continuous girders are more economical. For general purposes, however, the tendency has been towards the adoptions of unit spans freely supported at the ends and erected by understaging or other special means. It is of interest to note however that continuous girders can be employed with advantage to facilitate erection in difficult places over deep gorges or navigable rivers. Further it may be noted that the cantilever type of bridge employed for very large spans (for example the Forth Bridge in Scotland consisting of 1700 feet spans) is an adaptation of the principle of continuous girders applied in a suitable way to a very large span.

For the purposes of the present chapter it is proposed to consider the design of unit lattice spans up to 350 feet of the "through" type which will normally be found in the plains of India.

The different types of web systems employed for lattice girders will be discussed subsequently under "Framing the girder".

45. **Rules for Design and Calculation.**—The external forces which may be in action on any span have been enumerated and described in paragraphs 11 to 18, and in dealing with the design of plate girder spans reference has been made to those external forces apart from dead and live loads, namely impact and wind which invariably require consideration. It is now proposed to give in full the rules for calculating wind pressure and the other external forces which were formerly only briefly described and which are of relatively greater importance in larger spans.

The following rules are quoted from the Government of India Railway Department Rules as proposed by the President of the Indian Railway Bridge Committee in 1921.

46. **Wind Pressure.**—(*A*) *Wind pressure on unloaded bridges* :— Unloaded bridges up to 300 feet span must be capable of resisting a wind pressure of 40 lbs. per square foot. Above this length a special

determination of wind pressure per square foot must be made, depending on thelocal conditions. Bridge trestles must be capable of resisting a wind pressure of 56 lbs. per square foot.

(*B*) *Wind pressure on loaded bridges*:—In the case of a loaded bridge the intensity of wind pressure may be assumed to be not greater than that necessary to overturn a wagon; say 40 lbs. per square foot for the 5 feet 6 inch gauge, 30 lbs. per squre foot for the metre gauge and 25 lbs. for the 2 feet 6 inch and 2 feet 0 inch gauges.

The additional pressure on the leaward rail is to be taken into consideration.

The total area upon which the wind pressure on a loaded span acts is the nett area of the train surface above the rail for the highest vehicles *plus* the nett area of the superstructure exposed above the top of the train and the nett area below the rails.

The nett area of the superstructure above the top of the train and below the rails must be multiplied by the following constants to allow for the effect of two girders.

(*a*) Plate girder deck spans with a solid deck ... ... 1·00

(*b*) Plate girder dick spans with open deck between the girders and plate girder through spans ... ... 1·25

(*c*) Open web girder spans with solid decking ... ... 1·50

(*d*) Open web girder spans with open decking also trestle piers ... . 1·75

In the case of double track bridges each of the above constants is to be increased by 0·25.

47. **Temperature Stresses**—Stresses due to change of temperature are to be computed for a range of temperature of 100 degrees Fahrenheit or for such greater range as the circumstances of the case render necessary. The coefficient of expansion per degree Fahrenheit may be taken as ·000006.

48. **Traction Loads**—Traction and brake loads are to be taken as a horizontal force acting at the rails of each track in the direction of the moving train, and are to be computed as one-seventh of the vertical train load (*see* Table II).

49. **Centrifugal Force**—The centrifugal force due to a train travelling on a curve should be computed as that fraction of the train load given by the formula $\frac{V^2}{15\,R}$ in which V is the velocity of the train in miles per hour and R is the radius of the curve in feet.

50. Alternating Stresses.—Members subjected to alternating tensile and compressive loads shall be designed as struts capable of resisting the greater load *plus* half the less, except in the case of wind bracing which may be designed to resist only the greater load. The rivetted end connections of members subject to reversal shall be designed with sufficient rivets to resist the sum of both the loads.

The existence of reversal in the web system of a girder is illustrated in Fig 12. The curves A and B in the figure represent the live load

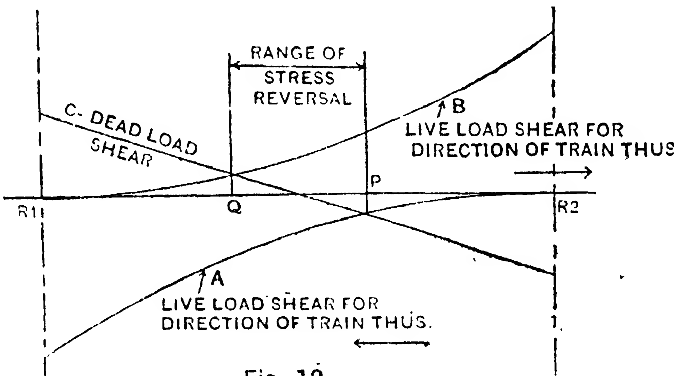


Fig. 12.

shear diagrams for a train moving in opposite directions. The line C is the dead load shear diagram. Where this line intersects the two live load curves draw verticals through P and Q; then the space between P and Q in the range of reversal of stress in the web system. Consider the train moving from right to left, then the total shear at any point is the sum of the ordinates of the diagrams A and C of live and dead load respectively. At any point beyond the middle of the span the dead load shear becomes negative and at the intersection of A and C the total shear is zero. It is clear that for all points between the maximum at the end and the point P the shear stress is of one sign. By similar reasoning if the train passes from left to right, the shear between R_2 and Q is of one sign but opposite to that between R_1 and P. Hence the range PQ is subject to reversal of stress.

I may be observed that such reversal not only occurs when a train comes on in the opposite direction but it occurs also when the same train is passing off the span, and it will be apparent that in the case of a single engine or heavy load concentration passing at high speed over a span, the reversal of stress is extremely rapid.

51. Framing the Girder..—The outline of a girder is generally framed according to a certain type. Girders are frequently described according to the shape or run of the booms, whether parallel or bowed, etc. This feature, however, is of less fundamental importance than the triangulation of the web system. The following features will have to be decided in framing the girder :—

(*a*) "Through" or "Deck" span.

(*b*) Depth of girder at the centre.

(*c*) Depth of girder at ends in the case of "bowstring" girders.

(*d*) Number of panel points.

(*e*) Type of web system.

Item (*a*) is determined by consideration of headway underneath the bridge. As a rule the general elevation of the railway or roadway will be fixed by the nature of the surrounding country and there is a limited amount of grading up that can be economically resorted to. If, therefore a certain minimum headway under the bridge is required for the passage of flood waters, craft, or vehicles in the case of a roadway, and at the same time the surrounding country is flat as in the plains of India, a "through" type of bridge is invariably adopted. In a "through" bridge the load passes between the girders and the depth from rail or road surface to the underside of the girders is the least possible. On the other hand where a bridge crosses a deep gorge or valley and there is no limitation of headway under the bridge it is common to adopt a "deck" type of span for spans not exceeding 300 feet. There is a certain economy in this as it is frequently possible to lighten the floor system if carried on the top of the main girders as is the case in plate girder deck spans where the floor system consists only of sleepers. In the case of large spans, however, the main girders have to be placed at a certain minimum distance apart to ensure lateral stability in the horizontal plane and it is, therefore, not possible to dispense with cross girders and floor system the same as is required in "through" spans.

Item (*b*) in modern girders is fixed at a considerably greater proportion of the span that is possible in plate girders. In lattic girders the depth at the centre may be as much as one-seventh or one-fifth of the span. This makes for economy in the booms and reduces deflection. The saving in the booms is at the expense of longer web members. In recent designs of 100 ft. lattice girder spans where overhead bracing has been adopted, the depth of the girders is one-fifth of the span.

Item (*c*) is subject to practical and structural limitations. On theoretical grounds the ideal lattice girder would be shaped like a segment of a parabola or semi-ellipse in which case the depth at all points would approximate to the bending moment and the boom section would be constant. The web system would be highly economical since the length of the members towards the ends where they are heaviest would be reduced to a minimum. In practice, however, the ideal can only be partially realized on account of the necessity of providing overhead bracing between the girders at the ends and the necessity of fixing this end or portal bracing at the certain minimum height or structure clearance above road or rail level. In the case of the 345 feet spans of the Hardinge Bridge shewn on Plate no. 1, the depth of the girders at the first panel point where the portal bracing occurs is 31 feet 6 inches. The minimum fixed structure was formerly fixed at 14 feet 6 inches but is now 16 feet 6 inches. If to this is added the depth of the floor system and the portal brace the very minimum height that can be adopted for any span is about 22 feet 6 inches. Hence it is evident that there is a strict limitation to the reduction of the end height, and accordingly, for spans of 100 and 150 feet the booms are generally made parallel. Some authorities adopt a parallel boom for spans of 200 feet as well. In the case of 100 feet spans like the new 100 feet "through" girders over the Ganges Canal at Roorkee, the girders are not a sufficiently deep to permit of any overhead bracing at all. In a "poney" type girder of this kind the torsional rigidity of the girders is secured by a specially rigid attachment of the cross girders to the vertical members of the main girders. The booms are made parallel for simplicity of construction.

Item (*d*) fixes the number of subdivisions or panels in the girder. The panel points are the points of concentrated loading where the cross girders are attached. In general the pannel spacing is controlled by the depth of the girder in order to preserve an effective inclination approximating to 45 degrees in the angle of the diagonal members of the web system. In case of a "bowstring" girder the depth varies and if the panel points are to be equally spaced so as to ensure uniformity in the floor system, it follows that the inclination of the diagonal members will vary with the height of the panel. In girders of this type it is usual to make the panel length equal to the end height or nearly so. This gives an inclination of 45 degree in the end diagonals and an increasing inclination towards the centreof the span. It also fixes the number of panels in the span.

In girders of large span it is found that the panel length fixed in this way, which will generally be about one-tenth of the span, has to be considered from the point of view of the economical framing of the floor system, since the panel length of the main girder fixes the length of the stringers or floor beams between the cross girders. Supposing a girder of 350 feet span is sub-divided into 10 panels, the length of each panel is then 35 feet and therefore the floor system of the span will now consist of 10·35 feet spans supported by the main girders. As the weight of a span increased at a rate approximately proportional to the square of the span, it becomes desirable to consider the possibility of sub-dividing these long panels into half or third panels in order to reduce weight in the floor beams. This problem has a direct bearing on the type of web system adopted.

The type of web system, *Item* (*e*), has been subject to many stages in evolution and practice has varied in different countries. If reference is made to paragraph 5 and figure 6, the student will observe that the mean slope of the lines of principle stress in the web system of a girder is at 45 degrees to the centre line of the beam, and it would appear that the most natural system of web bracing is a trellis system of tension and compression members at right angles and inclined at 45 degrees to the centre line of the girder. This system of bracing has been very largely employed and is still largely employed on the Continent of Europe. It has many practical advantages in rigidity, etc., but is subject to the theoretical disadvantage of being indeterminate, since engineers are not sure what proportion of the load is taken by the tension members and what proportion by the compression members. On the other hand the very indeterminacy of the system has been regarded by some engineers notably Sir Benjamin Baker who designed the Forth Bridge as a safeguard in that if one member failed there was another to take its load. In the practical design of unit spans however this type of bracing has been modified by making the compression members into vertical posts and making the tension diagonals strong enough to take the whole of the shear component. The vertical posts provide a convenient fixing for overhead bracing and stiffen the girder at the points of attachment of the cross girders. In the original adoption of this N type of construction, of which there are many examples in India to-day, the indeterminacy of the trellis system was not however eliminated. With the object of securing economy in the floor systems by sub-division of the panels of the girder it was found necessary to adopt a multiple intersection web system known as the whipple truss. The feature of this truss is two or more systems of web members overlapping. An important example of a girder of this description is found in the Dufferin Bridge at Benares (*see* illustration) in which a triple system of triangulation is adopted. In calculating the loads on the web members of a girder of his description it is assumed that the total shear at any panel is shared and equally divided in the three systems which are then separately calculated. It is found that in the maintenance of these trusses trouble is experienced with the diagonal members on account of non-uniformity of stress distribution resulting in excessive loading of individual bars and other bars becoming buckled and out of action. For this reason the whipple truss has now given way to the simple N truss with rigid members and, in the case of large spans, with sub-panelling or a theoritically determinate system. The Harding Lower Ganges Bridge illustrated in plate No. I is an example of the modern type of web system for lattice girders. The effective span is first divided into 11 main panels by a primary system of N bracing; it is then sub-divided into 22 half panels by a system of secondary triangulation consisting of a diagonal strut and sub-post. The slightly inclined member connecting to the centre of the main post is not an essential part of the triangulation, being provided to stiffen the main posts which, acting as struts, are of considerable length. This type of web system has the advantage of being determinate. It has a certain draw-back in that it induces secondary stresses in the main members due to the slight elastic extension or compression under load of the sub-members. In the Hardinge girders this is counteracted by arranging a slight eccentricity of thurst at the top boom connection which has the effect of neutralising the secondary stresses in the web members.

52. **Stress Sheets**—Having selected the type and framed the girder by the various considerations of the last article, stress sheets have to be prepared which shew for every member the maximum load to which it will be subjected in service. For those members which will be subject to reversal of stress, the maximum stress of both signs will be indicated.

The determination of the stress in a framed structure under a static or stationary system of loading is a comparatively simple problem and the ordinary principles of graphic statics apply. The members of a bridge, however, are subject to the varying effect of a moving load and for every member there is some position of the moving load which gives the maximum effect in that member. In the case of railway bridges the problem is further complicated by the fact that the equivalent uniformly distributed live load which we use for the purpose of design to represent the effect of a train has a varying intensity per foot run depending on the length of the bridge covered by the load. The heaviest train consists of two loco-motives followed by loaded vehicles and the maximum possible concentration of loading per foot run is greater on a short span which gets the full effect of the loco-motive driving wheel than on a long span where the effect is spread out. For the same reason the impact allowance is a higher percentage on a smaller span. Therefore, for different positions of a train on a bridge the unit load per foot run and the impact factor vary. In the Tables appended to this chapter are given the equivalent uniform loads for all spans from 0 to 500 feet in both bending moment and shear for the normal standard of loading now in use on Indian Railways (Standard B) of 1903 *plus* 25 per cent).

The determination of the maximum stresses is a relatively simple matter in the case of the booms which are proportioned to resist the maximum bending moment at any position on the span. The maximum bending moment for all positions occurs when the span is fully loaded. Having made a computation of the dead weight of the structure, the uniformly distributed live load for the full span in taken from the table. The impact effect is then added and the total equivalent vertical loading on the span obtained. From the total distributed load the parabola of maximum bending moment is obtained. The effect of lateral wind pressure must also be investigated to determine its effect on the booms. If, however, the direct tensile or compressive load in the booms due to lateral wind force does not exceed 25 per cent. of the total of the stresses due to vertical loading the effect of wind may be neglected. Having determined the maximum bending moment for all panels of the span the total bending moment is divided by two to give the effect on each girder. The boom stress at any panel is then obtained by dividing the bending moment by the effective depth of the girder. In cases of "bowstring" girders the effective depth at any panel is determined by

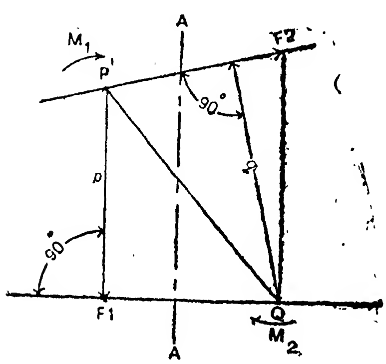


Fig. 13

the method of sections, *see* Fig. 13. Let A-A be a section through any panel. First take moments about P. If M_1 equals the bending moment about P. then for equilibrium we have—

$$F^1 \times p = M_1$$

$$F_1 = \frac{M_1}{p} \quad \ldots \ldots \ldots \quad (1).$$

which gives the stress in the bottom boom.

Now take moments about Q. By the same reasoning.

$$F_2 = \frac{M_2}{q} \quad \ldots \ldots \ldots \quad (2).$$

where "q" is the perpendicular distance of the point Q from the top boom.

The maximum stresses in the web members of a paralled girder occur when the front of the load has reached that position in the panel under consideration in which it divides the panel length in the same proportion as it divides the span. A simple construction for giving the position of the front of the load for all panels is shewn in Fig. 14. From the two

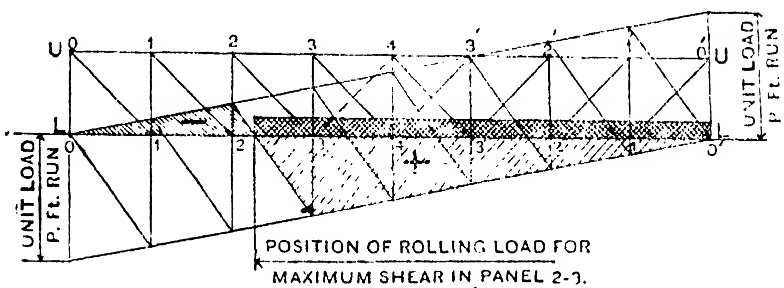


Fig. 14.

ends of the span set off equal distances on the verticals above and below the base line respectively. Join the ends of the base line to the opposite perpendicular as shown. Where these two parallel sloping lines intersect the verticals through the panel points, draw diagonals as shown. It will be apparent that by similar triangles, the intersections of the diagonals with the base line fulfil the condition above and give the positions of the front of the load for the maximum effect in the various panels. Taking now each panel individually the loaded length is measured off and the unit shear load for this length ascertained from the tables. The impact factor for this length is also calculated and live load shear for the panel determined as in paragraph 22. This represents the vertical live load reaction, to his must be added the dead load shear and the total vertical force in he panel obtained. We can now apply the principles of statics to the problem to determine the stress in the diagonal. If the booms are parallel, this vertical force must be balanced by the vertical component of the diagonal stress, hence the latter is obtained by the triangle of forces. If the booms of the girder are not parallel however, we apply the method of sections and consider the static equilibrium of the portion of the girder to the left of the section. Referring to fig. 15 suppose it is required to determine the

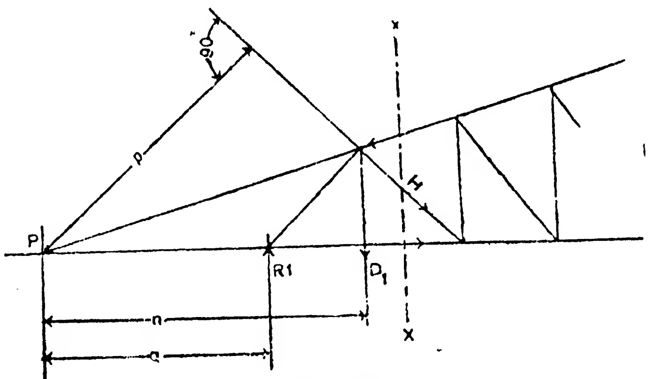


Fig. 15.

tension H in the second diagonal say. Imagine the girder cut at the section X-X and produce the line of the top boom member to cut the horizontal line of the bottom boom at P. Produce diagonal H and draw a perpendicular on to it from P. Now we know the reaction R_1 which is the dead load reaction *plus* the live load reaction. Consider the forces acting on the portion of the girder to the left of X-X. First we have the forces in the top and bottom booms and the diagonal. Then we have the reaction R_1 and the dead weight D_1 at the panel point. Of these forces we know R_1 and D_1, and our aim is to determine H. Now consider the equilibrium of the forces by taking moments about P. Since the top and bottom booms intersect in P they have no moment about P, and we have by considerations of statical equilibrium.

$$H \times p = R_1 \times q - D_1 \times n_1$$

$$\text{hence } H = \frac{R_1 \times q - D_1 \times n_1}{p} \qquad \text{...(1)}$$

It will be observed that if we are considering any other panel the same result applies except that we deduct the additional dead load panels weights multiplied by their respective distances from P

$$\text{i.e. } H = \frac{R_1 \times q - D_1 \times n_1 - D_2 \times n_2 - \text{etc.}}{p} \qquad \text{...(2)}$$

53. **Fixing the scantling.**—The minimum nett area of the various scantlings is determined by the maximum permissible unit working stress divided into the total maximum stress to which members are subjected. The fixing of the scantling of the various members is controlled very largely by experience. Bearing in mind that all members shall be rigid, the members will be generally built up in box section. Booms are generally constructed with two side plate connected to a flange. The open side is usually stiffened up by small lattice bars and the section is stiffened at intervals by diaphrams. Referring to plate No. 1 of the Hardinge girders the sections of the various members are shewn to an enlarged scale and the dimensions given. It will be noted that the neutral or centre of gravity line of the sections of the booms is at a certain distance from the flange plate which distance must be kept constant throughout the girder. As the section of the boom increases towards the middle of the span, it is important that the extra thicknes or plates added should not alter the position of the centre of gravity of the section. For this reason increase of metal is obtained by adding extra thickness to both the sides and the flange of the boom. The importance of this feature lies in the fact that in order that the direct loading in the member may be uniformly distributed over the section, it is essential that the line of application of the force should coincide with the centre of gravity of the section. This applies to all members of a framed structure and the sections of scantlings must be located in accordance with this principle about the axial lines of the frame or structure. Neglect to observe this principle results in producing bending or secondary stresses in the members apart from the direct stresses to which they are subjected.

The web members of a girder are commonly built up of four angle bars with or without a web plate. In larger members a box section is used. Web scantlings are usually symmetrical with the exception of the raking end post which usually has the same form as the top boom. The heavy raking end struts are required to take the diagonal component of the maximum end shear are also subjected to transverse secondary or bending stress due to the transmission of the upper wind loads through the portal bracing down to the bearings of the girder.

The jointing of the members of a framed structure is a problem of practical design. Members are generally connected by means of gusset plates. The axes of all intersecting members must pass through a common point. This follows from the primary assumptions made in the statical computation of the stresses. The rivetting of the end connections of members must be carefully designed so that firstly the shear strength of the rivets shall equal to or in excess of the requirements of dynamic loading, and secondly that the groups of rivets shall be distributed symmetrically about the axis of the section.

The floor system of a span is subject to the same computations as described for plate girders. The effective span taken as a basis of live load concentration is equal to twice the distance between the cross girders. Thisgives the maximum reaction at any cross girder. The proportioning of the floor members of a lattice spans is subject to rather different rules from the proportioning of free plate girders. That is to say the depth of the cross girders and floor beams is made considerably greater in proportion to length than is the case in ordinary plate girders. It is here not a question of economy only but the necessity of providing great stiffness in the floor members. The advantage of this is that the deflection of the floor members under load is reduced to a negligible amount and the deleterious effect of such elastic deformations on the cross girder connections to the main girder, etc. is avoided.

Having designed the main girders and floor system of the span it remains to consider the provision of suitable systems of bracing between the two girders so as to convert the whole system into a right unit capable of resisting heavy leteral wind forces and the dynamic racking and traction forces due to the passage of the live load. In a "through" lattice span there are four distinct systems of lateral bracing between the girders. The various systems of bracing are illustrated on plate No. 1 of the Hardinge girders. First there is the lower lateral wind or traction bracing which occurs in the plane of the bottom boom. This takes the horizontal shear forces due to the action of a side wind on the lower part of the girders and also on the surface of a train whilst on the track. Next there is the top lateral wind bracing which takes the wind shear due to the wind on the top half of the girders. Next there is the so called sway bracing in the vertical plane between each pair of post which serves primarily to stiffen up and maintain the alignment of the girders. Finally there are the two portal braces at the ends of the span the use of which has already been described.

Conclusion.—The subject of steel bridges may be extended to cover the consideration of larger cantilever, suspension and special types of structures including large roofs, steel frame buildings, etc. all of which are of practical importance and come within the province and the principles of bridge design. It is not intended however to go into the subject at greater length as it is rather the province of the specialist. In what has been presented to the student an attempt has been made to describe as clearly as possible the principles and the guiding features which control the design of simple girders, bridges laying stresses on the essential points. The development of analysis by influence lines and the calcuation of secondary stresses are subjects which have been briefly mentioned, but which space will not permit of developing in the scope of this Manual. The principles given if thoroughly grasped by the student will enable him to examine intelligently any bridge which in his practice as a Civil Engineer he may be called upon to do and to deal if necessary with the design of simple spans.

THE END

CURVE TABLES

Rules for the Design and Inspection of Railway Bridges

TABLE I—STANDARD (B + 25%) LOADING

STANDARD EQUIVALENT UNIFORM LOAD IN TONS PER FOOT OF EACH TRACK. TO BE USED FOR CALCULATING BENDING MOMENTS

Effective span in feet	5′–6″ Gauge		Metre Gauge		Effective span in feet	5′–6″ Gauge		Metre Gauge	
	Total load	Load per foot	Total load	Load per foot		Total load	Load per foot	Total load	Load per foot
3	45·0	9·000	25·0	5·000	60	157	2·616	109·4	1·823
6	45·0	7·500	26·6	4·433	62	161	2·596	112·2	1·809
7	45·0	6·429	28·3	4·036	64	165	2·579	114·9	1·795
8	45·0	5·625	29·9	3·735	66	169	2·561	117·7	1·783
9	45·0	5·000	31·5	3·500	68	173	2·544	120·4	1·770
10	45·0	4·500	33·1	3·313	70	177	2·529	123·1	1·759
11	49·4	4·489	34·7	3·159	72	181	2·514	126·0	1·749
12	53·1	4·428	36·4	3·031	74	185	2·500	129·0	1·739
13	56·5	4·386	38·0	2·923	76	189	2·486	131·0	1·730
14	59·8	4·268	39·6	2·830	78	193	2·474	134·0	1·721
15	62·8	4·184	41·3	2·750	80	197	2·463	137·0	1·711
16	65·5	4·094	42·9	2·680	82	201	2·451	139.0	1·701
17	68·1	4·008	44·5	2·618	84	205	2·440	142·0	1·690
18	70·6	3·924	46·1	2·563	86	209	2·429	144·0	1·679
19	73·1	3·849	47·8	2·514	88	213	2·418	147·0	1·669
20	75·5	3·776	49·3	2·463	90	217	2·408	149·0	1·660
21	77·8	3·703	50·9	2·423	92	220	2·398	152·0	1·650
22	80·0	3·636	52·5	2·386	94	224	2·388	154·0	1·640
23	82·1	3·571	54·1	2·353	96	228	2·378	156·0	1·630
24	84·2	3·510	55·6	2·318	98	232	2·369	159·0	1·621
25	86·4	3·455	57·1	2·285	100	236	2·360	161·0	1·613
26	88·5	3·404	58·8	2·260	105	245	2·335	167·0	1·588
27	90·6	3·356	60·4	2·236	110	254	2·313	172·0	1·564
28	92·7	3·311	62·0	2·214	120	272	2·270	183.0	1·523
29	94·8	3·268	63·5	2·190	130	291	2·235	193·0	1·485
30	96·8	3·225	65·0	2·166	140	308	2–200	204·0	1. 454
31	98·7	3·185	66·5	2·145	150	325	2·166	214·0	1·425
32	100·8	3·149	68·0	2·125	160	341	2·133	224·0	1·399
33	102·8	3·114	69·5	2·106	170	357	2·101	234·0	1·375
34	104.8	3·081	71·0	2·089	180	373	2·073	244·0	1·354
35	106·8	3·050	72·5	2·071	190	389	2·046	254·0	1·335
36	108·8	3·021	74·0	2·055	200	404	2·021	264·0	1·319
37	110·8	2·994	75·5	2·040	210	420	1·998	274·0	1·304
38	112·8	2·968	77·0	2·026	220	434	1·974	284 0	1·290
39	114·8	2·943	78·5	2·013	230	449	1·953	294·0	1·278
40	116·8	2·919	80·0	2·000	240	464	1·933	304·0	1·265
41	118·7	2·896	81·5	1·988	250	478	1·913	314·0	1·255
42	120·8	2·875	83·0	1·976	260	493	1·896	324·0	1·245
43	122·8	2·855	84·5	1·965	270	508	1·880	334·0	1·236
44	124·7	2·835	86·0	1·955	280	522	1·865	344·0	1·228
45	127·0	2·818	87·5	1·945	290	537	1·853	352·0	1·215
46	129·0	2·801	89·0	1·935	300	552	1·841	361·0	1·204
47	131·0	2·785	90·5	1.926	325	589	1·811	385·0	1·186
48	133·0	2·769	92·1	1·918	350	625	1·786	410·0	1·171
49	135·0	2·753	93·5	1·909	375	662	1·764	435·0	1·159
50	137·0	2·738	95·0	1·900	400	698	1·744	459·0	1·148
52	141·0	2·711	98·0	1·883	425	734	1·726	484·0	1·139
54	145·0	2·685	100·8	1·866	450	770	1·711	508·0	1·128
56	149·0	2·661	103.7	1·851	475	817	1·698	532·0	1·120
58	153·0	2·638	106·5	1·836	500	843	1·685	557·0	1·113

OTE—(*i*) For purposes of comparison the load equivalent to any train may be calculated either by the method given in the Bulletin of the International Railway Congress, August 1901 (also in Technical Section Paper No. 121) for moments at the *sixth* point of span, or by actual wheel loads.

(*ii*) For spans below 35 feet the maximum moment, wherever it occurs, is to be taken as the centre ordinate to the parabola of uniform load, as the girders will usually be of uniform section.

(*iii*) In this Table I. is the *effective*, not the clear span.

TABLE II—STANDARD (B+25%) LOADING

Standard equivalent uniform load in tons per foot of each track to be used for calculating Shears

L. Loaded length in feet which produces the maximum shear in the member under consideration

L. in feet	Load per foot: 5′ – 6″ Gauge	Load per foot: Metre Gauge	L. in feet	Load per foot: 5′ – 6″ Gauge	Load per foot: Metre Gauge
5	9·000	5·400	60	2·933	1·977
6	8·129	5·084	62	2·908	1·956
7	7·428	4·750	64	2·875	1·940
8	6·875	4·469	66	2·848	1·925
9	6·445	4·250	68	2·823	1·910
10	6·100	4·050	70	2·800	1·896
11	5·818	3·886	72	2·778	1·881
12	5·583	3·729	74	2·755	1·865
13	5·365	3·596	76	2·735	1·849
14	5·178	3·488	78	2·715	1·833
15	5·018	3·334	80	2·695	1·816
16	4·875	3·220	82	2·678	1·800
17	4·753	3·119	84	2·658	1·784
18	4·640	3·030	86	2·640	1·768
19	4·528	2·950	88	2·623	1·753
20	4·425	2·879	90	2·605	1·739
21	4·333	2·814	92	2·590	1·725
22	4·250	2·756	94	2·575	1·714
23	4·176	2·703	96	2·560	1·700
24	4·103	2·654	98	2·545	1·688
25	4·030	2·608	100	2·530	1·675
26	3·963	2·565	105	2·498	1·648
27	3·898	2·526	110	2·466	1·621
28	3·840	2·490	120	2·410	1·576
29	3·785	2·456	130	2·360	1·538
30	3·733	2·425	140	2·313	1·504
31	3·685	2·396	150	2·271	1·475
32	3·640	2·368	160	2·233	1·419
33	3·598	2·343	170	2·199	1·426
34	3·560	2·318	180	2·166	1·405
35	3·523	2·296	190	2·139	1·385
36	3·485	2·274	200	2·111	1·369
37	3·453	2·254	210	2·088	1·354
38	3·420	2·234	220	2·065	1·340
39	3·388	2·216	230	2·044	1·328
40	3·358	2·199	240	2·024	1·315
41	3·328	2·183	250	2·006	1·304
42	3·298	2·166	260	1·990	1·294
43	3·272	2·151	270	1·974	1·285
44	3·248	2·136	280	1·959	1·276
45	3·223	2·123	290	1·945	1·268
46	3·198	2·110	300	1·931	1·260
47	3·173	2·098	325	1·903	1·244
48	3·150	2·085	350	1·878	1·229
49	3·130	2·073	375	1·855	1·216
50	3·110	2·061	400	1·835	1·205
52	3·068	2·041	425	1·818	1·195
54	3·030	2·021	450	1·801	1·186
56	2·995	2·004	475	1·786	1·179
58	2·968	1·988	500	1·774	1·171

TABLE III—STANDARD (B+25%) LOADING

STANDARD CROSS GIRDER REACTIONS IN TONS

Note—(*i*) The table printed below is derived from the Load specified in Table I according to the formula

$R \frac{2M}{S}$ where S spacing of the cross girders in inches.

M maximum bending moment on the stringer, considered as of span 2 S.

R reaction at middle point of said stringer.

(*ii*) Half of the reaction given in this Table is to be considered as applied at each point where a rail-bearer rests on the cross girder.

Distance apart of Cross Girders, Centre to Centre in feet	Reaction in tons	
	5′-6″ Gauge	Metre Gauge
8	32·8	21·5
9	35·4	23·1
10	37·8	24·7
11	40·0	26·3
12	42·1	27·8
13	44·3	29·4
14	46·4	31·0
15	48·4	32·5
16	50·4	34·0
17	52·4	35·5
18	54·4	37·0
19	56·4	38·5
20	58·4	40·0
21	60·4	41·5
22	62·4	43·0
23	64·5	44·5
24	66·5	46·0
25	68·5	47·5
26	70·5	49·0
27	72·5	50·4
28	74·5	51·9
29	76·5	53·3
30	78·5	54·7
31	80·5	56·1
32	82·5	57·5
33	84·5	58·9
34	86·5	60·3
35	88·5	61·6

APPENDIX A

Values of co-efficient C in Kutter's formula for different H. M. Depths and co-efficients of roughness

Slope	Co-efficient of roughness f	H. M. D. r. 1	2	3	4	6	8	10	15	20	30	50
1 in 10,000 = ·0001	·020	67	81	89	94	102	107	111	118	122	128	134
	·025	52	64	71	76	84	88	92	98	102	108	114
	·030	42	53	59	64	70	75	78	85	89	95	100
	·035	35	45	51	55	61	66	69	75	79	84	91
1 in 5,000 = ·0002	·0 0	63	82	89	94	100	105	108	113	117	122	126
	·025	54	64	72	76	82	87	89	95	95	103	108
	·380	44	54	59	63	69	73	76	82	85	89	94
	·035	37	45	51	55	60	64	67	72	76	80	85
1 in 2,500 = ·0004	·020	70	88	89	94	99	108	107	111	115	118	123
	·025	55	65	71	76	81	85	88	92	96	99	104
	·030	45	54	59	63	69	72	75	79	83	86	91
	·035	37	45	51	55	60	63	66	70	73	76	82
1 in 1,000 = ·001	·020	71	88	89	93	91	102	105	109	113	116	120
	·025	56	66	71	75	81	84	87	90	94	97	101
	.030	45	54	59	63	68	71	74	78	31	84	89
	·035	38	46	51	54	59	63	65	69	72	75	79
1 in 100 = ·01	·020	72	83	89	93	99	102	105	109	112	115	119
	·025	56	66	71	76	81	84	86	90	93	96	100
	·030	45	55	59	68	68	71	74	77	80	83	87
	·035	38	4[illegible]	51	55	59	61	65	68	71	74	78

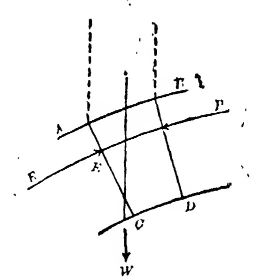
APPENDIX B

Calculation of stability and thickness of voussoir arch rings symmetrically loaded with a vertical load.

The theoretical calculation of the stability of arch rings involves some of the most difficult problems that occur in engineering, but there is a simple graphic method by means of which the stability can be calculated by a series of approximations. This method is described below.

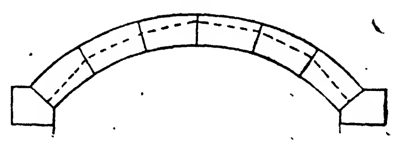
Let A B C D be a voussoir of an arch. The load on the voussoir consists of its own weight and the weight carried on the extrados between the dotted lines A B. Let W be the combined weight acting through the common centre of gravity. The voussoir is also subjected to the pressure P of the voussoir above it, and to the reaction R of the voussoir below it. Equilibrium is maintained by these three forces and if we know W and P we find R and also its point of application E, which is the centre of pressure for the joint A C.

Fig. 1.



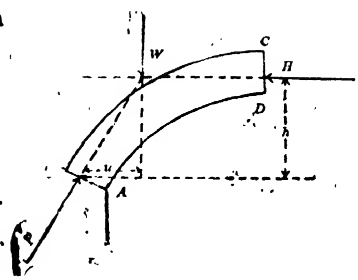
Now we know that for stability the point E must lie inside A C and that, moreover, to prevent tension in the mortar, it must lie within the middle third of the joint. For purposes of calculation the arch may be considered as divided up into a number of imaginary voussoirs as shown in *Fig.* 2 and we may suppose that the centre of pressure at each joint has been found, and all the centres of pressure have been joined together by a roken line. If the number of voussoirs is very large, the lines joining the centres of pressure will hardly be distinguishable from a curve. This curved line is called the **line of resistance**, and from it we can obtain the point of application (centre of pressure), and direction of the pressure at any section of the arch. For safety, it must lie within the Middle third of the arch ring.

Fig. 2.



The line of resistance also gives the direction of the resultant pressure of any joint of the arch ring, this direction being a tangent to the curve. If this direction makes an angle with the normal to the joint greater than the angle of repose of the material of which the arch consists, the arch will fall by sliding, but, as a matter of fact this inclination is seldom, probably never, sufficient to produce failure in this manner except in straight arches.

Fig 3



The problem then is to find the line of resistance. It will be seen from *Fig.* 3 that the line of resistance in a symmetrical arch uniformly loaded is horizontal at the crown C ; at this point therefore the pressure is horizontal. If we suppose the right half of the arch removed the left half is kept in equlibrium by :

(1) The load it carries (W).

(2) The horrizontal pressure H at C.

(3) The pressure P at the abutment A.

Now a force is determined when we know (1) its direction, (2) its point of application, and (3) its magnitude. W is easily determined. Of H we, only know the direction and of P we know nothing except that it is equal and opposite to the resultant of W and H. The problem cannot, therefore, be solved until we know the point of application and he magnitude of H. To obtain these, it is necessary to adopt what is known as Moseley's **Principal of least Resistance** which is as follows :

" If the forces which balance each other in or upon a given body or structure be distinguished into two systems called respectively **active** and **passive** which stand to each other in the relation of cause and effect, then will the passive forces be the **least** which are capable of balancing the active forces consistently with the physical condition of the body or structure."

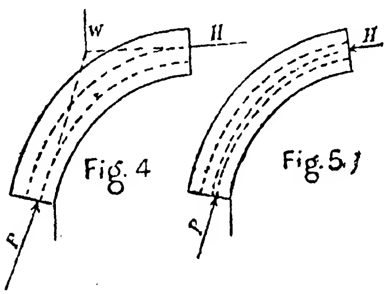
In the case under consideration W represents the active force and H and P the passive forces. It therefore follows that in the actual arch, the magnitude of H will be the least possible. To each value of H there is a corresponding line of resistance. Combining this condition with that of the position the line of resistance should occupy in the ring, we arrive at the result that the value of H must be the minimum that will give a line of resistance which is just included **within** the middle third of the arch ring. This line of resistance is called the line of **least resistance**.

To find the true line of resistance mathematically is difficult, even in simple cases of uniform vertical loading and symmetrical arches, but by the trial and error method above referred to, the line of least resistance can be found very readily, with sufficient accuracy for practical purposes. This method is as follows :

In *Fig.* 3, taking moments about the point of application of the force P we have

$$|\overline{W \times a}| - |\overline{H \times h}| = O.$$
$$\text{or } H = \frac{a}{h} W.$$

From this question it will be seen that H is least when a is least and h is greatest, which occurs when H is acting at the upper edge of the middle third of the thickness in the arch, and the point of application is the lower edge of the middle third at the skewback of the abutment as shown in *Fig.* 4. On the other hand the greatest permissible value of H will be when a is greatest and h least, i.e., when H is applied at the lower edge of the middle third and P at the upper. The true value of H will lie somewhere between the least and greatest values thus found, and is the least value of H, corresponding to the lowest position, that will allow of the line of resistance being just included within the middle third of the arch ring, *see Fig,* 5.



The line of resistance corresponding to any particular value of H can be found by the simple graphic method explained below, which will easily be followed by any student acquainted with elementary statics.

Suppose that the arch is divided into a number of voussoirs, each kept in equilibrium by three forces as shown in *Fig.* 1, and let the weight on each voussior acting through its C, G. be w_1, w_2, w_3, etc, Now beginning from the first voussoir next the crown, the three forces acting on it may be represented by this traingle in which H is the horizontal thrust at the crown, w_1 the load and P_1 the resulting pressure on the lower joint. Taking next the second voussoir, the forces keeping it in equilibrium are those shown by firm lines in this figure, the dotted lines representing the triangle of forces of the first voussoir. In this figure P_1 represents the thrust communicated

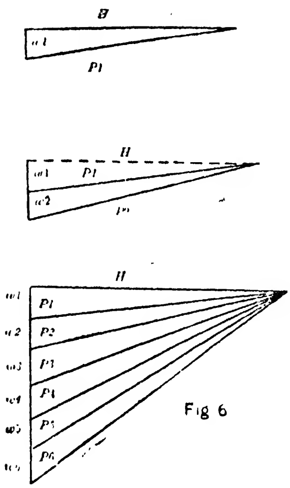


Fig 6

to the second voussoir by the first one, w_2 the load on the second voussoir and P_2 the thrust on the second voussoir. Proceeding in this way the diagram of forces acting on all the voussoirs from the crown to the abutment would take the form shown in *Fig.* 6, in which P_1 P_2 P_3 . . . represent the magnitude and direction of the pressures acting on the joints from the crown downwards to the abutment.

Let us now consider the particular arch shown in *Fig.* 7, the thickness of which has tentatively been fixed by the empirical rules given in Chap. V of the Manual.

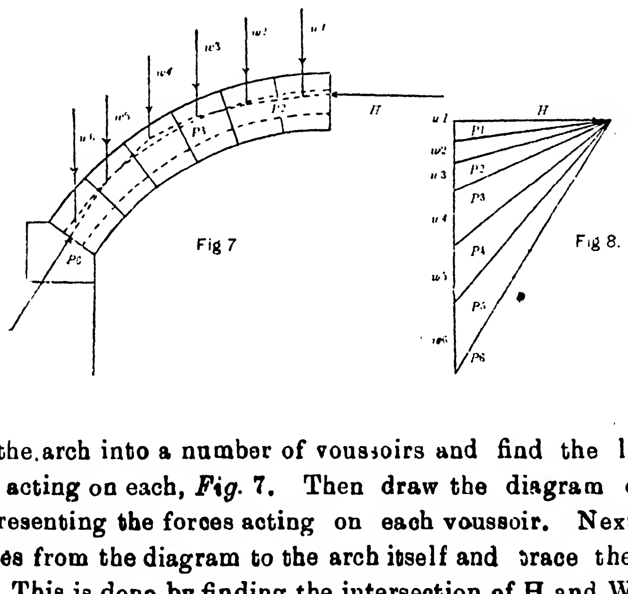


Fig 7 Fig 8.

Divide the arch into a number of voussoirs and find the laods w_1, w_2, w_3, etc. acting on each, *Fig.* 7. Then draw the diagram of forces *Fig.* 8 representing the forces acting on each voussoir. Next transfer the pressures from the diagram to the arch itself and trace the line of resistance. This is done by finding the intersection of H and W_1 in *Fi* . 7, and drawing P_1 parallel to the corresponding line on the diagram from the point of intersection of H and w_1 to the next load line w_2; from this point of intersection P_2 s drawn to w_3, parallel to the diagram line B_2, and so on. The line joining the several points of intersection is the line of resistance. It will be observed that the line of resistance in *Fig.* 7 is not included within the middle third of the arch ring, the reason being that the point of application of H has been chosen too high. The point of application must now be lowered, the effect of which will be to raise the value of H. A new diagram of forces and a new line of resistance is obtained for the altered conditions in a similar manner, and this is repeated until a line is found which just fits in within the middle third and is therefore the line of least resistance required. *See Figs.* 9 and 10.

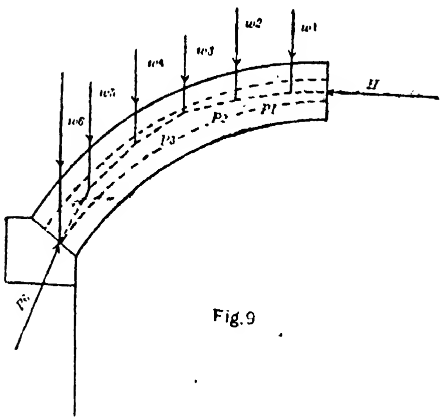


Fig. 9

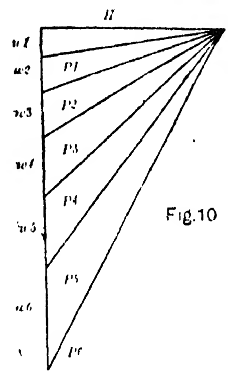


Fig. 10

When the line of least resistance has been found and the corresponding values and points of application of the pressures at different points the actual **maximum intensity of pressure** at any section of the arching can be found by the formula usually applied to uniformly varying pressures on a plane surface. $P \max = \frac{2F}{3D \times B}$ where F is the total forces acting on the joint, D is the distance in inches of the point of application of the force from the nearest edge of the arch ring, B is the width of the arch ring in inches normal to the span, and **P** (max). the maximum intensity of pressure per square inch.

APPENDIX C

Calculations of strength and dimensions of reinforced concrete slabs, girders and arches

Before discussing the calculations of stresses it is necessary to describe the rotation to be used and the strength of steel and concrete assumed for the purposes of calculation. The safe working stresses are those fixed by the Joint Committee of the R. I. B. A. in their second report of 1911.

w=load per unit length of span.

l=span.

W=total load.

c=safe compressive strength of 1 : 2 : 4 stone concrete in Portland Cement mortar ... 600 lb. per sq.

t=safe tensile strength of steel ... ... 16,000 " " "

s=safe resistance of steel to shear ... 12,000 " " "

E_c=Modulus of Elasticity of steel ...30,000,000 " " "

E_s— " " " " stone concrete ... 2,000,000 " " "

$m=\frac{E_s}{E_c}=15.$

a =adhesion of grip of cement concrete to metal ... ... 100 " " "

y =circumferential areas of tensile reinforcement.

= width of beam.

a = effective depth of beam, *i.e.* from extreme compression surface to tensile reinforcement.

b_1=width of table of T beam.

d_1=depth " " " "

k =ratio of depth of neutral axis from extreme compression surface to effective depth of beam, kd being the actual depth.

p =ratio of sectional area of tensile reinforcement to effective area of beam, pbd being the actual area = A_t.

p^1=ratio of sectional area of compressive reinforcement to effective area of beam, p_1bd being the actual area =A_c.

A = total area of section = bd.

M_r=moment of resistance.

M=maximum bending moment= { For supported beams, load uniform,= $\frac{Wl}{8}$

For fixed beams, load uniform, = $\frac{Wl}{12}$ at the ends and $\frac{Wl}{24}$ at the centre. But for beams imperfectly fixed, as they usually are in practice, and for beams continous, Mb may be taken to be $\frac{Wl}{12}$ both at the ends and at the centre.

Reinforced concrete beams are calculated on the assumption that in accordance with Hook's law (*ut tensio sic vis*) the following conditions hold within the elastic limits :

(1) strain (elongation or shortening) is proportional to the s tress,

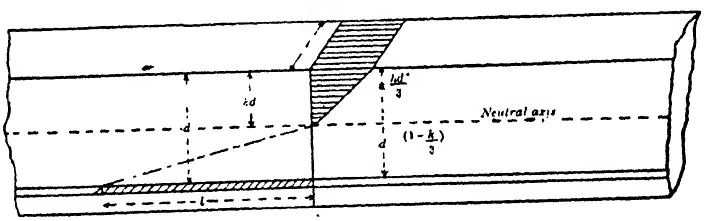
(2) stress and, therefore, strain are proportional to their distance from the neutral axis.

(3) strain varies inversely as the Modulus of Elasticity, or, in other words $E = \frac{\text{stress}}{\text{strain}}$. It follows from this that E_s being m times E^c at an equal distance from the neutral axis the stress in steel will be m times that in concrete. It is further assumed that the concrete below the neutral axis takes no tension which is taken up entirely by the steel reinforcement.

Rectangular beams with tension reinforcement

Now referring to *Fig.* 1 it will be seen that the stress in the concrete will vary uniformly from a point on the neutural axis to a maximum at the top as represented by the shaded triangle and the entire tensile stress will be uniformly distributed over the section of the steel.

Fig 1



From Fig. 1 it follows that the stress at the plane where the steel rod is placed, if the beam was homogeneous, would be to c at the top as $kd:d\ (1-k)$, but reinforced concrete being non-homogeneous, the stress in the steel is m times that in concrete so $c : \frac{t}{m} :: kd : (1-k)$ and

$$\frac{mc}{t} = \frac{k}{1-k}$$

From this we get $k - \frac{mc}{t+m_c}$

From this equation we can always fix the position of the neutral axis, knowing the stresses on concrete and steel. Using the values given in the above notation for m, c and t we get k—0·36.

Then again, from the above figure we see that the total compression on the upper side is $\frac{bc}{2}kd$ or $\frac{1}{2}kdcb$ and the total tension is tA_t or $tpdb$. As these two must be equal we have $\frac{1}{2}kdcb = tpdb$, from which we get $p = \frac{ck}{2t}$. From this equation, knowing k c and t we can find the ratio of reinforcement required. For the values assumed, the economical ratio will be ·00675.

For special steel or inferior concrete or broken brick, clinker or coke breeze other values must be assigned to m, c and t from which p and k can be calculated as above.

The moment of resistance of the beam will be the product of total compression × distance between c. g. of the compression prism and c. g. of tensile reinforcement or the product of total tension × the same distance...that is to say.

$$M_r = \tfrac{1}{2}kcbd^2\left(1-\frac{k}{3}\right) \text{ or } pbdt^2\left(1-\frac{k}{3}\right).$$

As M_r must equal M_b for equilibrium, the value of d can be easily found from the previously ascertained value of p and k and the known value of c or t. The value of M_b is ascertained in the ordinary way for the different conditions of load and the different methods of support; its value is given in the Notation for supported, fixed and continuous beams under uniform load.

To facilitate calculation, the general compression formula given above can be put thus:

$$M_b = \tfrac{1}{2}\;kcbd^2\left(1-\frac{k}{3}\right)$$

$$d = \sqrt{\frac{M_b}{\dfrac{k\left(1-\frac{k}{3}\right)cb}{2}}}$$

$$d = \sqrt{\frac{M_b}{C_c b}} \text{ where } C_c = \frac{k\left(1-\frac{k}{3}\right)c}{2}$$ values of which can be tabulated for different values of k corresponding to different values of c and t. See table below. d is thus easily found from the Bending Moment by taking the value of C_c from the table and assuming a suitable value for b. In the case of slabs take a width of 1 foot for purposes of calculation.

p can next be found from the formula $p = \frac{ck}{2t}$.

It is given in the table. From this the sectional area of reinforcement is at once deduced being pbd.

It will be seen that the values of k are based on m being 15. This will be practically correct for stone concrete 1 : 2 : 4 made portland cement mortar but if the quality of the concrete is very different its modulus of elasticity and therefore the value of m must be altered to meet the case.

Working stress of steel t	Working stress of concrete c	k=ratio depths of neutral axis assuming $m=\frac{E_c}{E_c}=15$ $k=\frac{mc}{t+mc}$	C_c = constant in equation $d=\sqrt{\frac{M_b}{C_c\, b}}$	p=ratio of steel in tension $=\frac{ck}{2t}$
16,000	400	0·272	49	·0034
	500	0·319	71	·0049
	600	0·360	95	·0067
	700	0·396	121	·0086
	800	0·428	147	·0107
14,000	400	0·300	54	·0043
	500	0·348	77	·0062
	600	0·391	101	·0083
	700	0·428	128	·0107

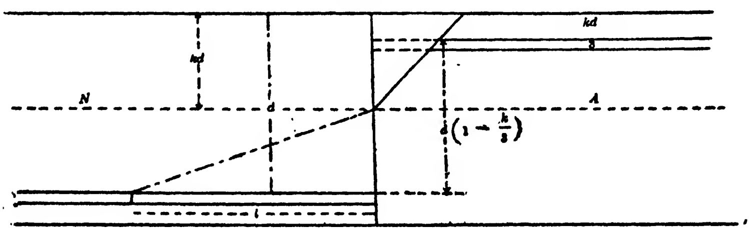
In a rectangular beam with tensile reinforcement only, if the ratio of reinforcement is greater than the economical ratio, the formula $M_b=\frac{1}{2}\,kcbd^2\left(1-\frac{k}{3}\right)$ should be used and if less, the formula $M_b=tpbd^2\left(1-\frac{k}{3}\right)$. If the ratio is the economical one, either formula may be used ; the latter is slightly simpler.

It will often be found that a beam designed as above is unsuitable, being too deep. If a shallower beam or girder is required to give more headway or for any other reason it may be obtained by increasing the tensile reinforcement beyond the economical ratio or by adding reinforcement in the compression fibres of the beam. In the former case, the value of p must first be fixed and from this value k must be found by the formula $k=\sqrt{p^2m^2+2pm}-pm$ which is derived by substitution from the fundamental formula given above, *viz.* $k=\frac{mc}{mc+t}$ and $p=\frac{ck}{2t}$. k having been found, the formula $M_b=M_r=\frac{1}{2}\,kcbd^2\left(1-\frac{k}{3}\right)$ must be used as p being greater then the economical ratio the strength of the beam is limited by the strength of the concrete. If compressive reinforcement is introduced, the case will require special treatment.

Beams with both compression and tension reinforcement

With compression rods, it is usual to give, in beams, the same covering of concrete on the compression side as on the tension; 1" to 1 ½", but to simplify calculation. it is assumed that the compressive reinforcement is placed at the centre of pressure of the compressive area of the concrete which will practically give a stronger result and be an error on the safe side.

Fig 2.



Under these conditions, the value of k remains as before; $k = \frac{mc}{t+mc}$ but the value of p will be different See *Fig.* 2.

Total compression $= \frac{1}{2}\, kctd + (m-1)\,\frac{2c}{3}\, ptbd.$

Total tension $= pbdt.$

Equating and simplifying $p = \frac{c}{t}\left[\frac{k}{2} + \frac{2}{3}(m-1)\,p'\right]$

$$\text{or } p' = \tfrac{3}{4} \times \frac{2pt - kc}{(m-1)c}$$

From the above we can find value of k and p or p' for any known value of m, c, t, i, and a given value of p' or p.

The general equation for the moment of resistance M_r of such a doubly reinforced beam will be.

$$M_r = \left(d - \frac{dk}{3}\right)\left(\frac{ck}{2} + \tfrac{2}{3}\, c\,(m-1)p'\right) bd.$$

Equating this to Mb the sectional area of the beam can be found. The equation will generally have to be used in double reinforcement cases, as a greater ratio than the economical is usually employed. In this case it is useful to know that the value of k in terms of $m\,p$ and p' will be

$$k = \sqrt{2pm + \left(\tfrac{2}{3}p'(m-1) + mp\right)^2} - \left(\tfrac{2}{3}\, p'(m-1) + mp\right).$$

The ordinary practice for finding p and p', is to assume suitable values for b and d and find p from the equation.

$$M_b = M_r = pbd^2t\left(1-\frac{k}{3}\right)$$

the value of k being $\frac{mc}{mc+t}$ and the values of m, c and t being known.

Having found p, p' is found from the formula.

$$p' = \frac{3}{4} \times \frac{2pt-kc}{(m-1)c}$$

If the resulting values of p and p' are unreasonably large or too small, the process should be gone through again with fresh values of b and d.

It should be carefully noted that there is fixed value of k for each ratio $\frac{c}{t}$, and each value of m, and also fixed corresponding values of the percentages of steel p and p'.

In using compression bars in a beam, it is necessary to remember that there will be a tendency in these bars to buckle and burst outwards through the concrete covering. To counteract this tendency it is absolutely essential that they should be tied in by stirrups or otherwise, either into the body of the concrete or to the lower reinforcing bars.

To find the deflection of rectangular beams, the following formulae may be used:

For supported beams-deflection $= \frac{5}{48}\ \frac{tl^2}{E_s\, d\,(1-k)}$

For fixed or continuous beams-deflection $= \frac{1}{32}\ \frac{tl^2}{E_s\, d\,(1-k)}$

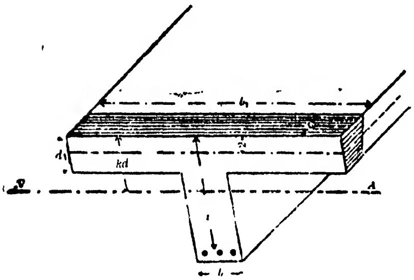
T beams with tension reinforcement

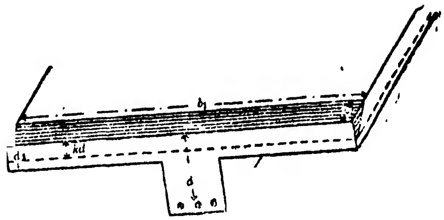
The Joint Committee of the R. I. B. A. recommend that the width of the slab or the upper flange of the T beam to be considered as forming part of the T beam must not be taken as greater than $\frac{1}{3}$ span or $\frac{3}{4}$ the spacing of these beams. For the main beams of bridges, it is perhaps safer to reduce this to some extent and make it half the spacing.

The width of the rib is generally made $\frac{1}{6}$ of b, or even a little larger if necessary to enable the reinforced bars to be correctly spaced (leaving $2\frac{1}{2}''$ intervals for concrete). The exact size of b, required to resist the horizontal shear at the junction of slab and rib, can be determined mathematically, and in important structures it is advisable to check this dimension by testing for shear.

T beams may be divided into 2 classes ; (1) in which the neutral axis falls at the lower edge of the slab or within it, (2) where the neutral axis falls outside the slab.

For the first class, the calculations are the same as those for rectangular beams except that the reinforcement is all grouped at the bottom of the rib of the **T** while the width taken for calculation is b, not b_1 the former being $\frac{b_1}{b}$ as a rule.





From the above figure it will be seen that the total compression above the neutral axis is, as in the case of rectangular beams, $\frac{1}{2}\,kdcb$, and that the total tension $= A_r\,t$ or pb_1dt. Also the compression on the upper side of the concrete and the tension in the reinforcement being proportional to their respective distances, from the neutral axis, we have as in rectangular beams

$$c : \frac{t}{m} :: kd : d\ (1 - k) \text{ or } k = \frac{mc}{t+mc}.$$

Furthermore, if we equate the compression and tension stress we get the same expression for p, *viz.*, $p = \frac{ck}{2t}$, p being the ratio of reinforcement to the total area b_1d (not bd). From these, as before, we get the moment of resistance $= \frac{1}{2}\,kb_1cd^2\left(1 - \frac{k}{3}\right)$. Alternatively, if less than the economical ratio of reinforcement is used, $M_r = pb_1d^2t\left(1 - \frac{k}{3}\right)$.

For the second class, the calculations are on exactly the same principle, but the formulae become more lengthy, owing to the expression for the trapezium representing compressive stress being more complicated than that for a triangle in the first class. Without going through all the intermediate processes of substitution and equation, the formulae corresponding to those of the first case may simply be stated as *follows* :—In this case the compression area is taken as only that of the slab portion. The small compression area in the rib between the slab and the neutral axis is neglected.

Total compression $= \frac{c\, b_1\, d}{2}, \left(\frac{2\, kd - d_1}{kd}\right)$

Total tension $=$ $A_t\ t$ or pb_1dt.

$k = \frac{mc}{t + mc} = 0{\cdot}36$ for values of m, c and t given in notation for 1 : 2 : 4 cement concrete.

The lever arm $= d - z$ (see *Fig.*) where $z =$ distance of centre of compression area from upper surface.

$$\text{The value of } z = \frac{d_1}{3} \cdot \frac{\frac{2c\,(kd - d_1)}{kd} + c}{\frac{c\,(kd - d_1)}{kd} + c} = \frac{d_1}{3} \times \frac{3kd - 2d}{2kd - d_1}.$$

Put $\gamma = \frac{d_1}{d}$. Then substituting γd for d_1 and $0{\cdot}36$ for k, we get

$$k - z = d - \frac{\gamma d}{3} \times \frac{3 \times 0{\cdot}36d - 2\,\gamma d}{2 \times 0{\cdot}36d - \gamma d}$$

$$= d\,\left(1 - \frac{1{\cdot}08\,\gamma - 2\gamma}{2{\cdot}16 - 3\gamma}\right).$$

Put $C_a = \left(1 - \frac{1{\cdot}08\,\gamma - 2\gamma^2}{2{\cdot}16 - 3\gamma}\right)$. The value of this can be calculated for various values of γ and tabulated for use. *See* column C_a in table further on.

Now mean stress in the concrete as shown above $= \frac{c}{2} \times \frac{2\,kd - d_1}{}$

$$= c - \frac{c\gamma}{2k}$$

$$= 600 - \frac{600\gamma}{0{\cdot}72}$$

Put this quantity $600\,\left(1 - \frac{\gamma}{0{\cdot}72}\right) = C_m$ and tabulate its values for various values of γ. *See* column C_m in table, further on.

We can now calculate the sectional areas of beam and of tension reinforcement quite readily from the Bending Moment.

$M_b = (d - z)\ C_m\ b_1\ d_1 = (d - z)\ C_m\ t\ A_t$

$d - z = d\ C_a$

$\therefore M_b = d\ C_a\ C_m\ b_1\ d_1 = d\ C_a\ t\ A_t$

We may illustrate the use of these formulae and the table by an example.

Example—A beam 20 feet span consists of a 4 inch slab with ribs 20″ effective depth, 8 feet apart centre to centre, carrying an external load of 112 lb. per square foot. Find sectional area of reinforcement and width of slab required to resist compressive stress.

Total load $= 8 \times 20 \times (112 + 50) =$ 25,920 slabs

Approximate weight of beam $=$ 180

Total weight $=$ 26,100 $=$ W

$M_b = \frac{26100 \times 20}{8} = 65{,}250.$

$\gamma = \frac{d_1}{d} = \frac{4}{20} = 0{\cdot}2.$

The lever arm $(d-z)$ will be found from the table to be $d \times$ $C_a = 20 \times 0{\cdot}91 = 18{\cdot}24$ inches $= 1{\cdot}52$ ft.

The area of steel required $= \frac{M}{dC_a\, t} = \frac{65250}{1{\cdot}52 \times 16000} = 2{\cdot}7$ sq. ins.

Now to get the area required in the compression flange or the width of flange required to act with 4″ depth of slab we have $b_1\, d_1 = \frac{M}{d\, C_a\, C_m}$ $= \frac{65250}{1{\cdot}52 \times 433} = 99.$

Now $d_1 = 4 \;\therefore\; b_1 = \frac{99}{4} = 24{\cdot}75$ inches.

—T beams with both compression and tension reinforcement—

We may sometimes have too little width in the flange and we must then supply the deficiency by steel in compression. This complicates the case still further. To simplify the complicated calculation for this condition as far as possible, we will assume that the values of m, c and t are as in the notation and that the value of k is therefore 0·36 and that $\gamma l - d_1$.

For reasons before explained, if steel is put on the compression side it will be stressed 15 times as much as the concrete in the same position. Assume as in the case of the rectangular beam that it is put in the centre of the compression area and that the lever arm is therefore the distance between the compression steel and the tension steel. Making $z =$ the distance between the top of the flange and the centre of the compression area, the intensity of stress in the steel at this point is obviously $c\ \frac{kd-z}{kd} 15 = 15 \times \frac{600(0{\cdot}30d - z)}{0\ 36d}$

The value of z is as given above $\frac{d_1}{3} \times \frac{3\ kd - 2d}{2\ kd - d_1}$. If in this expression we substitute for k, $0/{\cdot}36$, and for $d_1 \gamma d$ it will become $d\left(\frac{1{\cdot}08y - 2y_2}{2\ 16 - 3y}\right)$ and the value of the stress intensity of steel given above becomes

$$9{,}000 \left\{ \frac{0{\cdot}36 - \frac{(1{\cdot}08y - 2y^2)}{2{\cdot}16 - 3y}}{0{\cdot}36} \right\}$$

The value of this expression can be tabulated for different values of γ and it may be called $C\gamma$.

Now reverting for a moment to the calculation for a beam without compressive reinforcement, we found that the moment of resistance is equal to $A_t\ t\ (d-z)$ or, $b_1\ d_1 \times \frac{C}{2}\left(\frac{2\ kd - d_1}{kd}\right) \times (d-z)$ whichever is less. The first equation presents no difficulty. In the second equation there are three factors, $b_1\ d_1$ is the area of the concrete, $\frac{C}{2}\left(\frac{2\ kd-d_1}{kd}\right)$ is the mean stress in the concrete and $(d-z)$ is the lever arm.

As in the previous case, let us put C_m = the second factor $\frac{C}{2}\left(\frac{2\ kd-d_1}{kd}\right)$ or substituting γd for d_1, $\frac{C}{2}\left(\frac{2k-\gamma}{k}\right)$.

We then have $M_r = b_1\ d_1\ C_m\ (d-z)$.

The value of C_m is given in the table for various values of γ,

Now $\frac{M}{d-z}$ is the total longitudinal stress, tension in steel and compression in concrete. When $bd\ C\gamma$ is equal to or greater than $\frac{M}{d-z}$ no compression steel is required, but if it is less we must add steel. The stress of steel being 15 times that of concrete, if we add steel of section A to the compression flange, we get $M = (b\ d\ C\gamma + A,\ C_m)\ (d-z)$

from which we get $As = \left(\frac{\frac{M}{d-z} - b\ d\ C\gamma}{C_m}\right)$

The following table gives values of $C\gamma$ and C_m for a few values of γ which will be useful. These are calculated on the assumption that m= 15, c=600, t= 16000, k = 0·36.

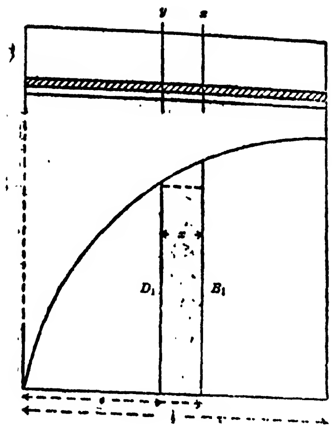
$\gamma = \frac{d_1}{d}$	C_a=ratio depth of lever arm	C_m=mean intensity of stress on concrete	$C\gamma$=intensity of stress in compressive steel
0·10	0·95	516	7,816
0·15	0·93	475	7,290
0·20	0·91	433	6,817
0·25	0·90	391	6,430
0·30	0·89	350	6,130
0·35	0·88	308	6,005

Shear Calculations

It is assumed that the shearing stress is uniformly distributed over the web of the beam which is taken to be the distance between the centre of the compression stress area and the centre of the tension reinforcement.

The horizontal component of the shearing stress in any short length of a beam under transverse strain may be represented by the difference in bending moments at the ends of that length divided by that length; and vertical shear is equal to horizontal shear at all points

Let B_1 and B_2 be the bending moments at the two sections X and Y of a beam and x the distance between them—Let a be the distance between the centre of the compression stress and the centre of the tension reinforcement, and S the total vertical shear at any section.



The difference of stresses due to B_1 and B_2 will be $\frac{B_1-B_2}{a}$. This will be communicated from the steel to the concrete by adhesion and produce a shearing stress in the concrete distributed over an area bx where b is the breadth of the beam. The unit shearing resistance of the concrete will thus be $\frac{B_1-B_2}{abx}$. If the safe shearing resistance of concrete (60 lb. per square inch) is less than this, steel shear reinforcement is necessary.

Let A =sectional area of steel reinforcement and s the safe unit shearing resistance of steel—we then have $A_s\, s + \left| bx \times 60 \right| = \frac{B_1-B_2}{a}$.

But, as above stated, the vertical shear in any short length of beam x may be represented by the difference of B_1 and B_2 divided by x—*i.e.* $S=\frac{B_1-B_2}{x}$ or $Sx=B_1-B_2$. From this equation S is easly found, as B_1 and B_2 the bending moments at any given distance from the end of the beam are known to be $\frac{we}{2}(l-e)$ foot pounds where w is the unit load per foot run, l is the span and e the distance of the section from the end. We thus have.

$$A_s\, s + 60\, bx = \frac{Sx}{a}, \text{ or, } As = \frac{(S-60\, ba)\, x}{as}$$

In important structures for greater security, the resistance of the concrete (60 bx) may be disregarded. If this is neglected, we have

$$A_s\, s = \frac{Sx}{a}, \text{ or, } A_s = \frac{Sx}{as}$$

From these two equations, the sectional area and spacing of the vertical shear member may be calculated. In the case of **T** beams, b should be taken to be the width af the rib.

For diagonal shear reinforcement at an angle θ to the horizontal.

$A_s \cos\theta$ should be taken $= \frac{(S-60\ ba)\, x}{as}$ or $\frac{Sx}{as}$

Adhesion stress$=\frac{S}{ya}$ which must not exceed 100 lb. per square inch for plain smooth bars, but for deformed bars, stresses up to 150 lb. per square inch may be allowed.

To find the correct spacing of stirrups or diagonal shear bars, use the following graphic solution given in the "Manual of Reinforced Concrete" by Marsh and Dunn.

Draw the bending moment diagram and divide the maximum ordinate of the curve into any suitable number of equal parts. Through these points draw lines parallel to the datum line of the curve cutting the curve. From the points where these lines cut the curve, drop perpendiculars to the datum line. The points where they cut the line will give the position of the stirrups. The stirrup where the maximum shear comes having been calculated the same material may be used for all the other stirrups. If diagonals are used in addition to stirrups, they must be taken into account at the points of maximum shear which will be the points of support in ordinary cases. The spacing of vertical stirrups should nowhere exceed the total depth of the beam.

Arches

For small arches up to 50′ span which may be considered as parabolic, under a load uniformly distributed over the whole span, the curve of pressures will be parabolic.

Let l=span.

r=rise, which is practically the same as the versed sine of the neutral curve.

w=load per square unit.

H=horizontal thrust.

R=thrust at springing.

$$H=\frac{w^{2}}{8r}$$

$$R=\frac{wl^{2}}{8r}\sqrt{1+\frac{16r^{2}}{l_1}}$$

Both the horizontal thrust and the reaction at the springing will act at the neutral line of the arch. The value of H and R being found the equations for columns under direct compression may be applied to obtain the sectional area of the arch at the crown and springing, allowing a pressure of say 30 tons per square foot.

In this case of parabolic or approximately parabolic arches, where the loading is not constant but may be taken to be uniform, the curve of pressure for the dead load may be assumed to follow the neutral line in the middle of the arch as before. The superimposed load may be assumed to cover half the span, as this loading causes the greatest bending moment. The curve of pressure for this may also be assumed as passing along the neutral line of the arch at the crown and the springing. This assumption will only be strictly correct for partial loading if the arch is hinged at crown and springing.

Taking w to be the dead load per square unit as before and putting w_1 for the superimposed load per unit.

$$H = \frac{l^2}{8r}(w + 2w_1)$$

$$R = \frac{l^2}{16r}(w+2w_1)\sqrt{\frac{1+4r^2(8w+4w_1)^2}{-l^2(w+2w_1)^2}}$$

The maximum bending moment produced by the superimposed partial load will be at a distance $\frac{l}{4}$ from the springing and will be $\frac{wl^2}{64}$. The thrust at this point will be $\frac{wl^2}{16r}(w+2w_1)$ sec γ, where γ is the angle made by the tangent to the neutral line with the horizontal at this point. To find what the sectional area of the rib should be, it will have to be calculated as a coloum under direct compression and transverse loading at this point.

These equations will only apply to arches hinged at the crown and springing if the weight of the arch and roadway can be considered as uniformly distributed. The ratio of reinforcement may be taken to be something between 1 and 1·5 per cent. distributed half near the intrados and half near the extrados.

For arches without hinges, the approximate depth of the arch at the crown may be found roughly by a formula devised from the study of existing arches by Mr. F. F. Weld and published in the "Engineering Record" of the 4th November, 1905.

$$d = \sqrt{l} + \cdot 1l + \cdot 005\, w_1 + \cdot 0025\, w.$$

where d = depth at crown in inches.

l = clear span in feet.

w_1 = exterior load in lb. per foot super. uniformly distributed.

w = weight of the dead load above the arch ring at the crown per square foot in lb.

The thickness of the arch ring at the quarter points should be $1\frac{1}{3}$ d.

The ratio of steel reinforcement may be taken to be ·0075 of the sectional area at the crown, the reinforcement near the intrados and extrados being generally equal.

APPENDIX D

Examples of plate and lattice girders showing their design and calculations of strength of the different members

EXAMPLE I

STEEL PLATE GIRDER RAILWAY BRIDGE FOR 5 FEET 6 INCHES GAUGE

(Plate A)

40 *feet clear Span*

General Specification

1. The bridge to be entirely of steel, except that iron may be used for service bolts, packing and washers.

2. The steel must be of such strength and quality as to stand the following tensional stresses, and to indicate the following percentages of elongation on fracture :

	Tensional stress per square inch	Percentage of elongation
	Tons	
Steel in plates, either with or across grain, and in angle or other bars—		
not less than	27	20
or more than	31	
Steel rods for rivets and bolts—		
not less than	25	25
or more than	28	

Side and end shearings must be taken as samples from every plate and flat bar and from as many angle and other bars as the Inspecting Engineer shall require. At least 20 per cent. of the samples so taken will be tested for tensile strength.

3. The working stresses not to exceed the following limits per square inch ; for tension, 5 tons ; compression, 4½ tons ; shearing, 6 tons ; and for bearing surface, 8 tons.

4. The length of span to be 40 feet clear, but for calculating the strength of the girder, to be considered as the length between centres of bearing on abutments.

5. The bridge to carry a single line of way, 5 feet 6 inches gauge and to have the rails placed directly on the upper flanges of the girders.

6. The live load to be as prescribed by the Government of India for 40 feet spans, that is, 2·7 tons per foot run of span.

7. The dead load to be the weight of steel in the structure, together with the weight of 18 sleepers, each 4·86 cubic feet at 30 lb. and two rails 75 lb. per yard.

8. The bridge to consist of two longitudinal plate girders, designed to resist the bending and shearing stresses, and each girder to be 44 feet in length. The web-plate to be stiffened at short intervals by vertical angle-irons. The girders to receive no lateral support from the abutment, and to be strongly connected to each other by 5 sets of vertical diagonal bracing spaced about equally over the span, as well as four sets of horizontal bracing attached to the top flanges. The rivets on the under side of the girders where these rest on the abutments to be countersunk.

9. The girders to have a camber of one inch.

10. All rivets in main angle bars of flange plates to be $\frac{7}{8}$ inch diameter and to be of steel, and all riveting to be done in the workshops of the manufacturer, except the joints and fastenings, which are to be made by turned bolts and nuts provided with hexagonal heads and $\frac{1}{4}$ inch washers.

11. All plates and bars must be carefully levelled and straightened before and after they are punched or drilled, and the edges of all plates must be planed dead true to dimensions.

12. All screwed bolts are to be screwed to Whitworth's standard thread, and no nut must fit too tightly to be turned by hand. The head and nuts (except those for timber, which are to be square) are to be hexagonal. The head and body of all the bolts are to be forged out of one piece.

Calculations

Data—

Clear span	40 feet
Effective span between centres of bearings	42 ,,
Gauge of rails	5 feet 6 inches.
Depth of girders, $\frac{1}{14}$th of span	3 feet.
Live load, per foot run	2·7 tons.
Dead load—	
Due to steel per foot run	·25 ,,
,, sleepers 18×4·86×30	2,624 lb.
,, rails, 2×25 per foot run	50 ,,

Limits of stress—Tension, 5 tons; compression, $4\frac{1}{2}$ tons; shearing, 6 tons; bearing surface, 8 tons per square inch.

Live load, 42×2·7	=113·4 tons.
Dead load—	
Steel, 42×·25	= 10·5 ,,
Sleepers, 2,624÷2,240	= 1·17 ,,
Rails (42×50÷2,240)	= ·93 ,,
Dead and live load on two girders	= 126·00 ,,
,, ,, ,, one girder	= 63·00 ,,

Stresses on flanges—The stress on flanges at centre of span is obtained by the formula $S = \frac{W \times L}{8D}$, where S = stress; W = distributed load; L = length of span; D = depth of girder. Then

$$S = \frac{63 \times 42}{8 \times 8} = 110 \cdot 25 \text{ tons.}$$

The sectional areas required in the tension and compression flanges will be—

Upper flange in compression, $\frac{110 \cdot 25}{4\frac{1}{2}} = 24 \cdot 5$ square inches.

Lower flange tension, $\frac{110 \cdot 25}{5} = 22 \cdot 05$ square inches.

To provide for these we use for—

Upper flange, 2 angles $4 \times 4 \times \frac{3}{8}$ ...	= 9·218	square inches.		
2 plates $16 \times \frac{9}{16}$ ...	= 18·00	,,	,,	
Total ...	27·218	,,	,,	
	Net area			
Lower flange, 2 angles $4 \times 4 \times \frac{3}{8}$	= 8·125	,,	,,	
2 plates $16 \times \frac{9}{16}$	= 16·03	,,	,,	
Total ...	24·155	,,	,,	

One of the flange-plates may be dispensed with from a point, let us assume $12\frac{1}{2}$ feet from the centre. The stress at this point is found by the formula $\frac{W}{2DL}(Lx - x^2) = S$, where W = distributed load, D = depth of girder, L = span, x = distance of point from abutment.

Then $S = \frac{63}{2 \times 8 \times 42}(42 \times 8 \cdot 5 - 8 \cdot 5^2) = 71 \cdot 18$ tons. The sectional areas required will be—

Upper flange, $\frac{71 \cdot 18}{4\frac{1}{2}} = 15 \cdot 81$ square inches.

Lower ,, $\frac{71 \cdot 18}{5} = 14 \cdot 23$,, ,,

The sectional areas left after removing one plate will be—

Upper flange, $27 \cdot 218 - \frac{18}{2} = 18 \cdot 218$ square inches.

Lower ,, $24 \cdot 155 - \frac{16 \cdot 03}{2} = 16 \cdot 14$,, ,,

We might likewise find at what point the remaining flange-plate could be stopped but the saving of material would be trifling, and greater rigidity will be given to the girder by its retension. It also provides a suitable attachment for the vertical and horizontal bracing.

To find the places where a plate may be discontinued in a flange, the most expeditious and exact method is to construct a Bending Moment Diagram and mark off at once on the curve the points where the ordinate representing the Bending Moment is equal to the product of the sectional area reduced by a plate multiplied by the working stress intensity multiplied by the depth of the girder. This graphic method will no doubt be familier to students who have done their Applied Mechanics course and will not be further explained here.

Shearing stresses—All the resistance to shear is supplied by the web and the rivets. The horizontal shearing stress is always equal to the vertical at every point of the girder. The vertical shearing stress is distributed over the whole vertical section—not uniformly it is true but nearly so. The vertical stress is a maximum at the ends of the girder and a minimum at its middle. Therefore, if the total vertical shear at the abutments where it is equal to the reaction (half load) is divided by the area of the vertical section of the girder and the quotient taken as the unit vertical and horizontal shearing stress at every point of the girder, the error, if any, will be on the safe side. The rivets are designed on this hypothesis.

Rivets.—The maximum shearing stress is at the end of the girder and is $63 \div 2 = 31{\cdot}5$ tons, giving $31{\cdot}5 \div 3 = 10{\cdot}5$ tons per foot run of the horizontal and vertical sections at that point. There must be in the horizontal and vertical joints of the web $10{\cdot}5 \div 6 = 1{\cdot}75$ square inches per foot run of shearing section in the plates and rivets; and $10{\cdot}5 \div 8 = 1{\cdot}31$ square inches of bearing area. The shearing area of a $\frac{7}{8}$-inch rivet is $\cdot 6$ square inch, and its bearing area in a $\frac{7}{16}$ inch plate is $\frac{7}{8} \times \frac{7}{16} = \cdot 38$ square inch. In a vertical joint of the web then, we will require $1{\cdot}31 \div \cdot 38 = 3{\cdot}4$ rivets per foot run, that the bearing area may be sufficient; and as the rivets shear in two sections, there will require to be $1{\cdot}75 \div 2(\cdot 6) = 1{\cdot}45$ rivets per foot run, that the shearing area may be sufficient. Taking four rivets to the foot, there will be left in the plate $(12 \times \frac{7}{16}) - (\frac{7}{8} \times \frac{7}{16} \times 4) = 5{\cdot}25 - 1{\cdot}53 = 3{\cdot}72$ square inches of shearing section per foot run, which is more than sufficient.

We proceed now to consider the horizontal joint between the angles and the boom. In this there will also be required $1{\cdot}31$ square inches of bearing area, and $1{\cdot}75$ square inches of shearing area per foot run, and there are two rivets (one in each angle) in the width, each rivet shearing at one section. The bearing of the rivet in the angle is in this case $\frac{7}{8} \times \frac{5}{8} = \cdot 54$ square inch. There will be required $1{\cdot}31 \div \cdot 54 = 2{\cdot}4$ rivets per foot run to give the necessary bearing area, and $1{\cdot}5 \div {\cdot}6 = 2{\cdot}9$ rivets to give the shearing area, or $1\frac{1}{2}$ rivets per foot run to each angle. In practice it will be more convenient to space the rivets closer than this. On referring to the drawing it will be seen that the pitch is 4 inches.

Joints in bottom boom—It will be observed that a joint occurs in one of the flange plates $9\frac{1}{2}$ feet from the end of girder. We previously found the stress at this point to be 71·18 tons. Of this the angles, which are unbroken, will transmit a part proportional to their area. The net section of the broken plate being 8·015 square inches, the proportion of the stress to be carried by the joint will be $(8{\cdot}015 \div 16{\cdot}14)$ $71{\cdot}18 = 35{\cdot}34$ tons. There will be required, therefore, in the rivets on each side of the joint $35{\cdot}33 \div {\cdot}6 = 5{\cdot}89$ square inches of shearing area and $35{\cdot}34 \div {\cdot}8 = 5{\cdot}41$ square inches of bearing area. The shearing area of $\frac{7}{8}$ inch rivet being ·6 square inch, the least number of rivets required will be $5{\cdot}89 \div {\cdot}6 = 10$ nearly on each side of the joint. The bearing area of a $\frac{7}{8}$ inch rivet on a $\frac{9}{16}$ inch plate being ·49 square inch, there will be required $4{\cdot}41 \div {\cdot}49 = 9$ rivets to give a sufficient bearing surface.

The number of rivets provided should be in excess of those shown as required by the calculations.

On referring to the drawing, it will be seen that the joint is made by extending the outer flange-plate beyond the point where it might have been stopped. This is a better arrangement than attaching a separate plate.

Shear of web—The web plate is of uniform thickness throughout $\frac{7}{16}$. It has been shown above to be thick enough to provide sufficient bearing area for the rivets at joints to resist the shearing stresses. The calculations to show it is stiff enough as a "long pillar" to withstand the compressive stresses due to shear are of a simple nature and will not be taken up here, *see* Applied Mechanics Manual. The plate is thick enough.

No break occurs in the angles of the boom, these being rolled in lengths to suit the girder. A joint occurs in the web at the centre of span, but there is no shearing stress there. In practice it is always usual to cover the joint of web plate by a cover plate on each side.

The bracing which secures the two girders to each other is formed of two transverse angles running across the top and bottom booms, to which the diagonals of the vertical bracings are attached. The angles of the horizontal bracing are $3'' \times 3'' \times \frac{3}{8}''$. These are fastened at each side to the flange-plates and connected at the centre by a joint plate. The use of these is to keep the girders truly parallel to each other.

The other details of construction will be understood by an examination of the drawing and need no further description.

Example II

IRON LATTICE GIRDER BRIDGE FOR SECOND CLASS ROAD

(*Plate B*)

60 *feet clear Span*

General Specification

1. The bridge to be entirely of wrought-iron except the bed-plates which may be of cast-iron. The wrought-iron must be of such strength and quality as to be equal to the following tensional strains, and to indicate the following percentages of contraction of the tested area at the point of fracture and percentages of elongation :

	Tension stresses per square inch	Percentage of contraction per square inch	Percentage of elongation in length of 10 inches
	Tons		
Round, square flat bars under 6 inches wide ..	24	20	15
Angle, T-bars and flat bars 6 inches wide and upwards	22	15	12
Plates	21	10	8
Plates across grain	18	5	4

2. The working stress per square inch to be for tension 4½ tons and for compression 4 tons, for all work except the lattice bars, for which the stresses will be 4 tons for tension, and 2½ tons for compression; for shearing, the stress to be 6 tons, and for bearing surface 8 tons.

3. The length of span to be 60 feet clear, but for calculating the strength of lattice girders to be considered as the length between the centres of bearings on abutments.

4. The clear width of roadway, that is, the distance between kerbs, to be 12 feet.

5. The live load to be taken at 112 lb. per square foot of roadway ; and the cross girders to be designed to bear a gun carriage considered as a central load of 3 tons on each cross girder.

6. The dead load to be the weight of iron in the structure, and the metalling of the roadway, which is not to exceed 6 cwt. per foot run of bridge.

7. The bridge to consist of two longitudinal lattice girders having the roadway between them, and to be designed for the loads specified.

8. The roadway platform to consist of cross girders supported on the bottom booms of the main girders. The flooring to be of buckled plates ¼ inch thick and the width of each plate not to exceed 3 feet. These are to be riveted to the cross girders, and to longitudinal angle-irons at sides of each plate. A kerb plate of iron is to be provided at each side of the roadway.

9. The alternate cross girders are to be prolonged beyond the outside face of each lattice girder, and a support is to be made therefrom to the top boom of the lattice girder, to give it lateral stiffness.

10. The riveting of the lattice girders to be generally ¾-inch, except in lattice bars, in which the rivets must be proportioned to the stresses on them; in the cross girders the riveting will be ⅝ and in the floor plate ½-inch. All rivets are to be cup-headed at each end, and the heads are to contain not less than 1½ diameters of the rivet. Where necessary, for the division of the work for transport, the rivets are to be left out, but the holes must, in all cases, be drilled ready for riveting.

11. All plates, bars and angle-irons must be carefully levelled and straightened before and after they are punched or drilled. All dead edges of plates and the ends of all angle-irons and bars must be planed true to dimensions.

12. The underside of the bearing plates must be perfectly level and the rivets countersunk. All bed-plates are to be absolutely flat and the guiding edges planed and made truly parallel.

13. All bolts are to the screwed to the Whitworth standard thread, and no nut must fit too tightly to be turned by hand. The heads and nuts of all bolts to be hexagonal.

14. The bridge is to be erected in the workshops of the manufacturer. so that accuracy of fit and perfection of workmanship may be assured.

The whole of the iron-work, with the exception of the bolts and rivets, is to be scraped perfectly free from rust, scale and dirt and then brushed all over with boiling hot linseed oil. It is afterwords to be painted with two coats of good oil paint, the first being of red lead and the second of some approved colour.

Calculations

Data—	Feet
Clear span	60
Effective span between centres of bearings	62½
Clear width of roadway	12
Depth of main girders to centres of booms $\frac{1}{15}$th of span ..	4
Depth of cross girders	1
Distance apart of cross girders	4

Limit of stress per square inch, tension, $4\frac{1}{2}$ tons; compression, 4 tons; shearing, 6 tons; and for bearing surface, 8 tons. For lattice bars in tension, 4 tons; compression $2\frac{1}{2}$ tons.

I—*Cross Girders*

Dead load on one bay—

		lb.
Iron work, 12×4×32	.. ..	= 1,536
Metalling, 4×6×112	.. ..	= 2,688
	Total ..	4,224
		or 1·88 tons.

Live load on one bay—

Due to crowd, 12×4×112	.. ..	= 5,376
	Total dead and live load ..	= 9,600

The load due to a gun carriage is 3 tons at centre, which is equal to a distributed load of 6 tons. This is greater than the live load due to a crowd and must be adopted. The maximum dead and live load is therefore $6+1\cdot88=7\cdot88$ tons.

Stresses on boom at centre—

$$S=\frac{W\times L}{8D}=\frac{7\cdot88\times13\frac{1}{2}}{8\times1}=13\cdot25 \text{ tons.}$$

Sectional areas required—

Lower flange in tension, $\frac{13\cdot25}{4\frac{1}{2}}=2\cdot93$ square inches.

Upper flange in compression, $\frac{13}{4}=3\cdot25$ square inches.

Sectional area provided—

Tension flange, 2 A. I. $2\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}=3$ square inches net area.

Compression flange, 2 A. I. $2\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}=3\cdot46$ square inches.

The bearing and shearing area of the rivets at the ends may be calculated in the same way as in the previous example, and need not be repeated here. The minimum section of plate for the web that would be adopted in practice is $\frac{1}{4}$-inch, and for rivets $\frac{5}{8}$ at 4-inch pitch which would give more than sufficient area for the stresses in the girder.

II—*Main Girders*

Dead load—

This may be found approximately by the rule given on page 99.

$$\left(\frac{\text{Span}}{500}+\frac{\text{square root of space}}{100}\right)\cdot8=\left(\frac{62\frac{1}{2}}{500}+\frac{\sqrt{62\frac{1}{2}}}{100}\right)\cdot8=\cdot1632 \text{ tons}$$

per foot run, for both girders.

Then,

Weight of one girder $=(62\frac{1}{2}\times{\cdot}1682)\div2$ .. = 5·1 tons.

" " platform $(1{\cdot}83\div2)$ 15 .. .. = 13·72 "

18·82 "

Live load—

The area of roadway being $61\frac{1}{2}\times12=750$ square feet, and live load 112 lb. per square foot the live load on one girder will be $\frac{750\times112}{2\times2240}$ .. = 18·75 tons.

Total dead and live load .. = 37·57 "

Stresses on flanges at middle of girder—

$$S=\frac{W\times L}{8D}=\frac{37{\cdot}47\times62\frac{1}{2}}{8\times4}\quad\ldots\quad=73{\cdot}37\text{ tons.}$$

Sectional area required in flanges at middle of girder—

Upper flange in compression, $\frac{73{\cdot}37}{4}$ .. = 18·34 square inches.

Lower flange in tension, $\frac{73{\cdot}37}{4\frac{1}{2}}$ .. .. = 16·30 " "

Sectional area provided—

Upper flange 2 Ch. 1, $7\frac{1}{4}\times3\times\frac{1}{2}$ .. .. = 9·75 square inches.

2 plates $15\times\frac{3}{8}$ .. .. = 11·25 " "

21·00 " "

Net area

Lower flange 2 Ch. 1, $7\frac{1}{4}\times3\times\frac{1}{2}$ .. .. = 9·18 square inches.

2 plates $15\times\frac{3}{8}$ .. .. = 10·125 " "

19·305 " "

One of these flange-plates may be dispensed with, let us assume at $16\frac{1}{2}$ feet from the centre, the stress at this point will be $\frac{W}{2DL}(Lx-x^2)$

$$=\frac{37{\cdot}57}{2\times4\times62\frac{1}{2}}(62\tfrac{1}{2}\times15-15^2)=53{\cdot}53\text{ tons.}$$

Sectional area required in flanges—

Upper flange, $\frac{53{\cdot}53}{4}$ .. .. .. = 13·38 square inches.

Lower " $\frac{53{\cdot}53}{4\frac{1}{2}}$ .. .. .. = 11·89 " "

The sectional area left on removing one plate will be—

Upper flange, $21{\cdot}00-\frac{11{\cdot}25}{2}$ .. .. = 15·38 square inches.

Lower " $19{\cdot}305-\frac{10{\cdot}125}{2}$ .. .. = 14·248 " "

We come now to consider the strains on the web. The simplest method of obtaining these is by tabulating the strains produced by each weight separately. The total load on the girder being 37·57 tons, there is on each apex 37·57÷15, say 2·5 tons. Tracing the action of $\frac{W}{1}$ it is seen that in the bar $\frac{W}{1}$ *a* there is a load $W\frac{x}{l}=2\cdot5\times\frac{56}{60}=2\cdot33$ tons producing tension, and in the bars towards the right abutment, there is a load $W\frac{y}{l}=2\cdot5\times\frac{4}{60}=\cdot16$ ton producing compression in these bars, and so on for the other weights. (All bars inclined down *to* the nearest support are in compression, those inclined down *from* the nearest support tension.) In tabulating the strains only those bars which incline in one direction need be considered, for the others are strained similarly and equally.

In the following table + indicates compression and — tension. The numbers are the loads on the bars. For the strains, the loads must be multiplied by secant 45° = 1·4. The rows (maximum + and maximum —) are obtained by adding the + values together, and the — values together respectively in each vertical column. The row (uniform load) is the algebraical sum of the values in the vertical columns.

Diagram of loads

o p′	14 n	13 m	12 l	11 k	10 j	9 i	8 h	7 g	6 f	5 e	4 d	3 c	2 b	1 a	$W\frac{x}{l}$	$W\frac{y}{l}$	Loads
+ ·16	..	+ ·16	..	+ ·16	..	+ ·16	..	+ ·16	..	+ ·16	..	+ ·16	..	—2·33	2·33	·16	W 1
..	+ ·33	..	+ ·33	..	+ ·33	..	+ ·33	..	+ ·33	..	+ ·33	..	—2·16	..	2·16	·33	” 2
+ ·49	..	+ ·49	..	+ ·49	..	+ ·49	..	+ ·49	..	+ ·49	..	—1·99	..	—1·99	1·99	·49	” 3
..	+ ·66	..	+ ·66	..	+ ·66	..	+ ·66	..	+ ·66	..	—1·83	..	—1·83	..	1·83	·66	” 4
+ ·83	..	+ ·83	..	+ ·83	..	+ ·83	..	+ ·83	..	—1·66	..	—1·66	..	—1·66	1·66	·83	” 5
..	+ ·99	..	+ ·99	..	+ ·99	..	+ ·99	..	—1·19	..	—1·49	..	—1·49	..	1·49	·99	” 6
+1·16	..	+1·16	..	+1·16	..	+1·16	..	—1·33	..	—1·33	..	—1·33	..	—1·33	1·33	1·16	” 7
..	+1·33	..	+1·33	..	+1·33	..	—1·16	..	—1·16	..	—1·16	..	—1·16	..	1·16	1·33	” 8
+1·49	..	+1·49	..	+1·49	..	— ·99	..	— ·99	..	— ·99	..	— ·99	..	— ·99	·99	1·49	” 9
..	+1·66	..	+1·66	..	— ·83	..	— ·83	..	— ·83	..	— ·83	..	— ·83	..	·83	1·66	” 10
+1·83	..	+1·83	..	— ·66	..	— ·66	..	— ·66	..	— ·66	..	— ·66	..	— ·66	·66	1·83	” 11
..	+1·99	..	— ·49	..	— ·49	..	— ·49	..	— ·49	..	— ·49	..	— ·49	..	·49	1·99	” 12
+2·16	..	— ·34	..	— ·33	..	— ·33	..	— ·33	..	— ·33	..	— ·33	..	— ·33	·33	2·16	” 13
..	— ·16	..	— ·16	..	— ·16	..	— ·16	..	— ·16	..	— ·16	..	— ·16	..	·16	2·33	” 14
+8·12	— ·16	— ·23	— ·65	+ ·99	—1·48	—1·98	—2·64	—3·31	—4·13	—4·97	—5·96	—6·96	—8·12	—9·29	Maximum—		..
..	+6·96	+5·96	+4·97	—4·13	+5·31	+2·64	+1·98	+1·48	+ ·99	+ ·65	+ ·33	+ ·16	..	..	Maximum+		..
+8·12	+6·80	+5·68	+4·32	+3·14	+1·83	+ ·66	— ·66	—1·83	—3·14	—4·32	—5·63	—6·80	—8·12	—9·29	Uniform Load		..

End pillars

Each of the end pillars are posts in compression under a load of 18·78 tons. They should be designed as "long pillars fixed at the two ends." The bar used is a channel iron $7\frac{3}{4} \times 3'' \times \frac{3}{8}''$ this may be tested for sufficiency, thus—

Area of channel = $4\frac{7}{8}$ sq. in. .. Least diameter of channel 3 inches.

Length of pillar = 48 inches .. Working stress 4 tons per square inch,

Safe load by Gordon's formula $= \dfrac{4 \times 4\frac{7}{8}}{1 + \frac{1}{3000}(16)^2} = 18$ tons. The actual load is 18·78 which is safe enough as the working stress has been taken rather low.

Bars in tension

Bar	Load × 1·4 = Stress	Limit	Sectional area	Area provided
	Tons	Tons	Sq. inches	Sq. inches.
1 *a*	9·92 × 1·4 = 13·00	4	3·25	$2 (4\frac{1}{2} - \frac{7}{8}) \frac{1}{2} = 3·62$
2 *b*	8·12 × 1·4 = 11·36	4	2·84	$2 (4 - \frac{7}{8}) \frac{1}{2} = 3·12$
3 *c*	6·80 × 1·4 = 9·52	4	2·38	$2 (3\frac{1}{2} - \frac{3}{4}) \frac{1}{2} = 2·75$
4 *d*	5·63 × 1·4 = 7·88	4	1·97	$2 (3 - \frac{3}{4}) \frac{1}{2} = 2·25$
5 *e*	4·32 × 1·4 = 6·04	4	1·51	$2 (3 - \frac{3}{4}) \frac{3}{8} = 1·68$
6 *f*	3·14 × 1·4 = 4·39	4	1·09	$2 (3 - \frac{3}{4}) \frac{1}{4} = 1·12$
7 *g*	1·83 × 1·4 = 2·59	4	·64	$2 (3 - \frac{3}{4}) \frac{1}{4} = 1·12$
8 *h*	·66 × 1·4 = ·92	4	·23	$2 (3 - \frac{3}{4}) \frac{1}{4} = 1·12$

These bars are attached to the channels by three rivets in each. The shearing area of rivets required in bar 1 *a* is $\frac{13 \div 6}{2} = 1·08$ square inches and in 3 *c* $\frac{9·52 \div 6}{6} =$ ·79 square inch. The area of three $\frac{7}{8}$-inch rivets is 3 × ·6 = 1·8 square inches and of three $\frac{3}{4}$-inch rivets 3 × ·44 = 1·32 square inches. The bearing surface required on rivets in bar 1 *a* is $\frac{13·0 \div 8}{8} =$ ·81, and in bar 3 *c* $\frac{9·52 \div 8}{2} =$ ·50 square inch. The bearing areas of these rivets in a $\frac{3}{8}$-inch plate are $3 \times \frac{7}{8} \times \frac{3}{8} =$ ·98 square inch and $3 \times \frac{3}{4} \times \frac{3}{8} =$ ·80 square inch.

Bars in compression

Bars	Load × 1·4 = Stress	Limit	Sectional area	Area provided
	Tons	Tons	Sq. inches	Sq. inches
9 *i*	·66 × 1·4 = ·92	2½	·86	2 A. I. 2 × 2 × ¼ = 1·87
10 *j*	1·88 × 1·4 = 2·56	2½	1·02	2 „ 2 × 2 × ¼ = 1·87
11 *k*	3·14 × 1·4 = 4·89	2½	1·75	2 „ 2 × 2 × ¼ = 1·87
12 *l*	4·82 × 1·4 = 6·04	2½	2·41	2 „ 2 × 2 × ⅜ = 2·71
13 *m*	5·63 × 1·4 = 7·88	2½	3·15	2 „ 3 × 2 × ⅜ = 3·46
14 *n*	6·80 × 1·4 = 9·52	2½	3·80	2 „ 3 × 2 × ½ = 4·21
p'o	8·12 × 1·4 = 11·36	2½	4·55	2 „ $4 \times 2 \times \frac{7}{16}$ = 4·86

The rivets in these bars will be the same as those in tension bars.

Joints and bottom boom—Two joints occur in the channel irons of the booms. The net section of one is 4·59 square inches, and the proportional stress will be $4·59 \times 4\frac{1}{2} = 20·65$ tons. The shearing area being $20·65 \div 6 = 3·4$ square inches, the number of rivets required on each side of joint will be $3·4 \div ·44$, say 9 nearly. The bearing area of a ¾-inch rivet on a ⅜-inch plate is $\frac{3}{4} \times \frac{3}{8} = ·28$ square inch, and the number of rivets required will be $\frac{20·65 \div 8}{·28} = 10$ nearly.

For the joints in the plates of the boom, the net section of one plate being 5·062 square inches and proportional shearing stress $5·062 \times 4\frac{1}{2} = 22·17$ tons, the number of rivets required for shear will be $\frac{22·77 \div 6}{·44}$, say 9, and for bearing surface we require $\frac{22·77 \div 1}{·28}$, say 10.

Similar joints should be provided for the plates and channels of the upper boom, the butting edges of these not being depended on for resisting compression.

The general design and details of construction will be seen on the *Plate*. The channel irons of the upper boom are prolonged over the abutments, and are connected to two vertical plates stiffened by angle-irons and channel irons, which forms a strong pillar on the abutment. These pillars are at one end made fast to the bed-plates, and slide on them at the other end, thus providing for the expansion of the iron work.

The cross girders of the flooring rest on the upper edge of the channels of the lower boom to which they are riveted. The buckled plates (4′ × 2½′) span the distance between the cross girders, and are riveted to the upper flanges of these, and at the sides to the logitudinal angle-irons which extend through the full length of the flooring. The safe distributed load on a buckled plate is given by Molesworth as ·5 ton per square foot, and for 10 square feet each plate should therefore be able to support about 5 tons.

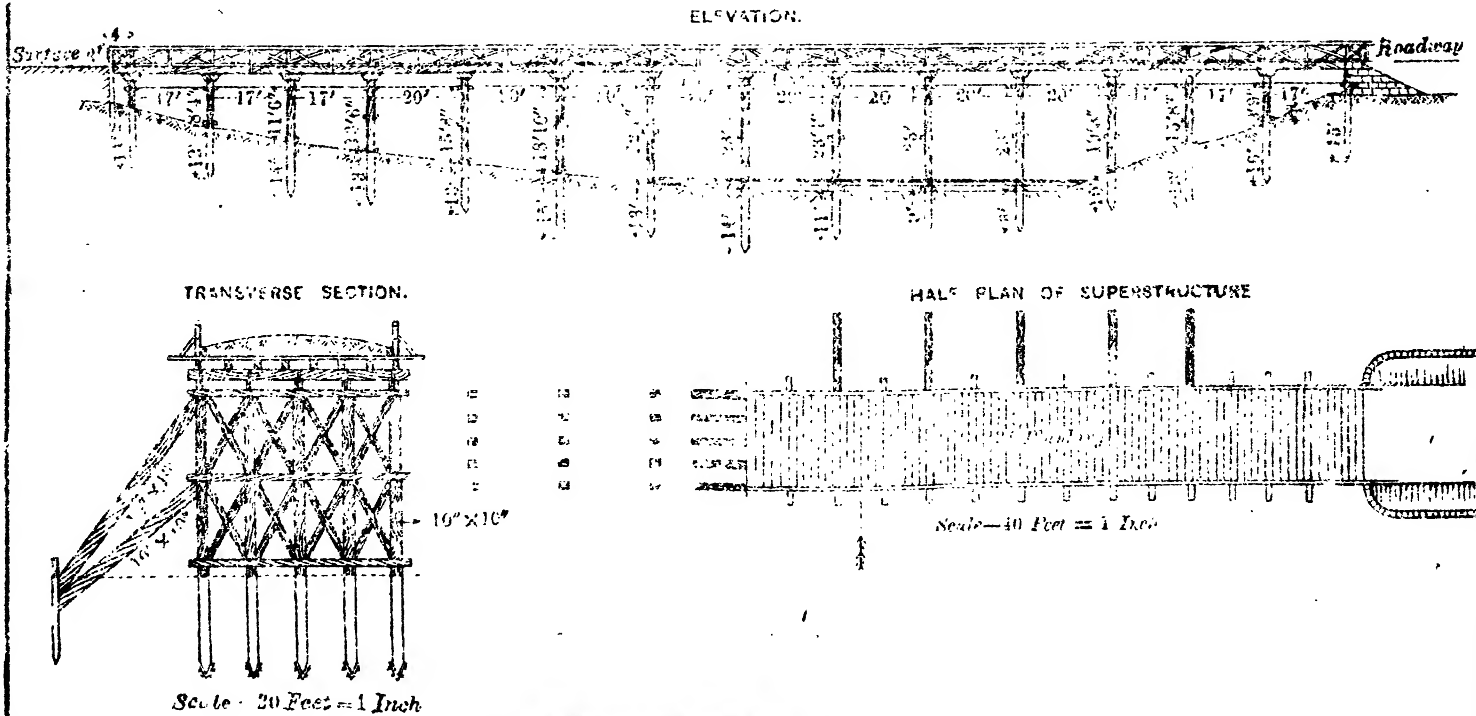
In the construction of the lattice girders a camber of 1¼ inches should be given. This is effected by making the spacing of the lattice bracing in the top boom slightly greater than in the lower boom.

PLATE I.

YANGUL BRIDGE.

Constructed on the Benhalla River,

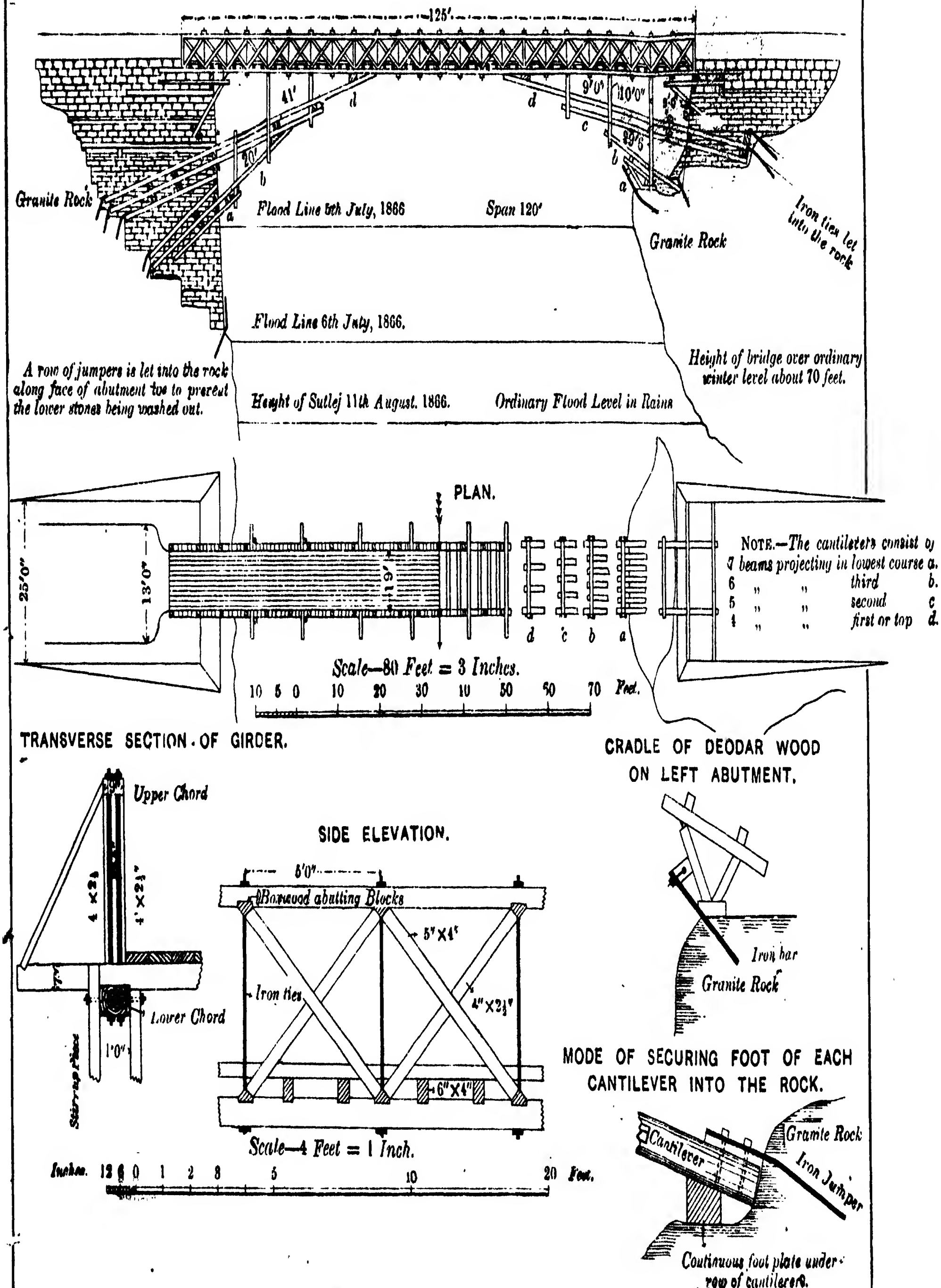
Dharwar District.



4.G.P.S-1947-P.S-U.P.(R)-1250

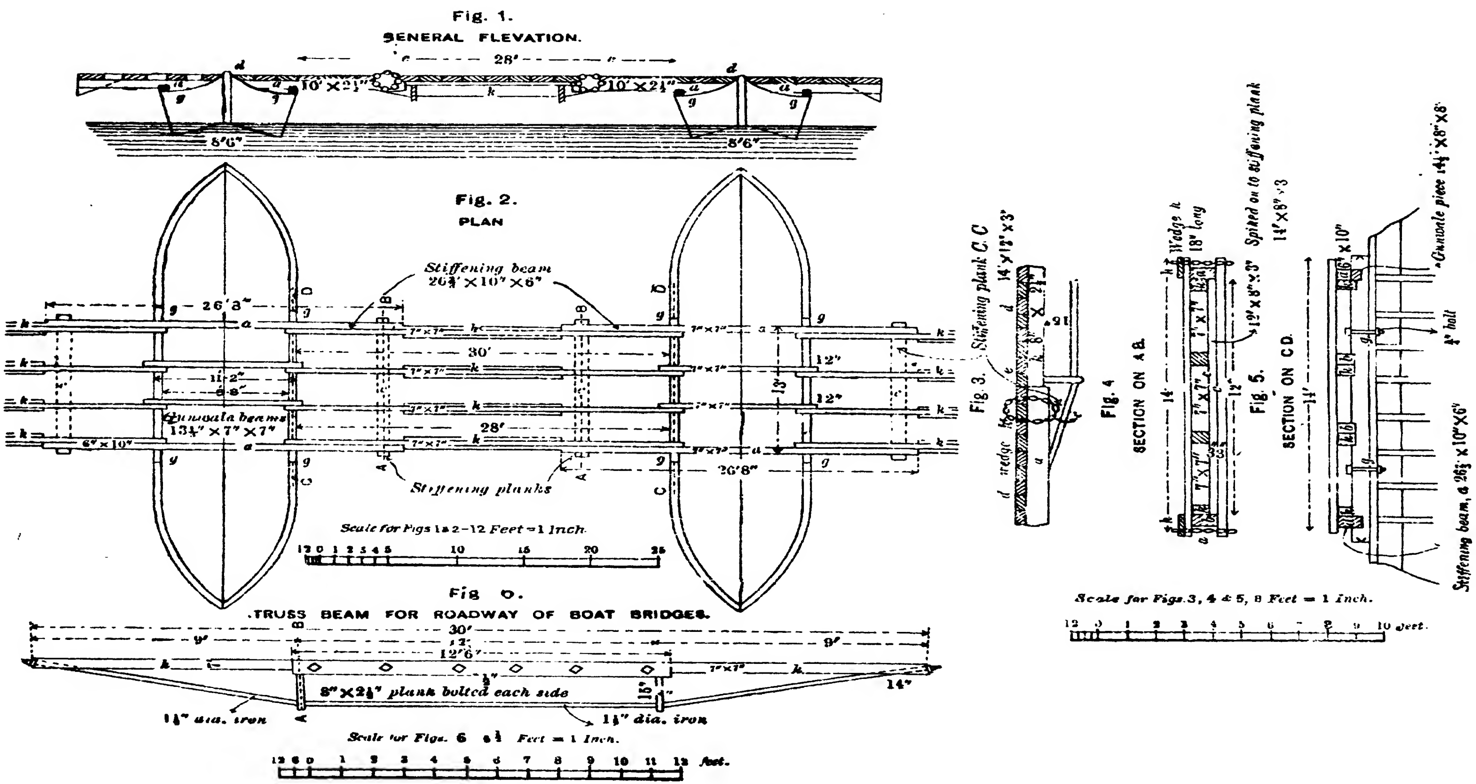
WANGTU BRIDGE OVER THE SUTLEJ.

Designed and built by Colonel Lang, *Royal Engineers.*



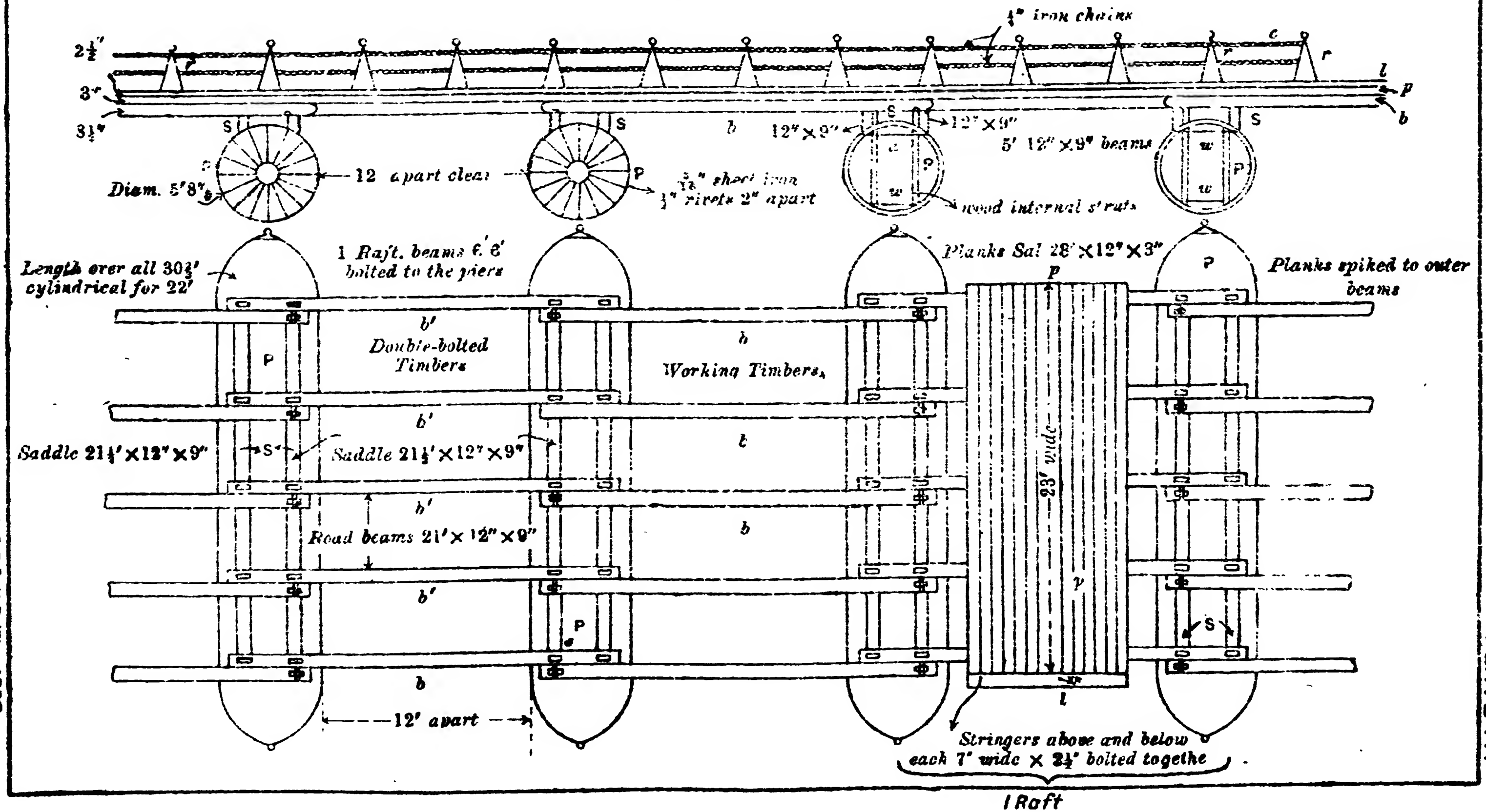
P.S.U.P (R) 4.S.P.S.1947.1250,

SUPERSTRUCTURE OF BOAT BRIDGES, PUNJAB.

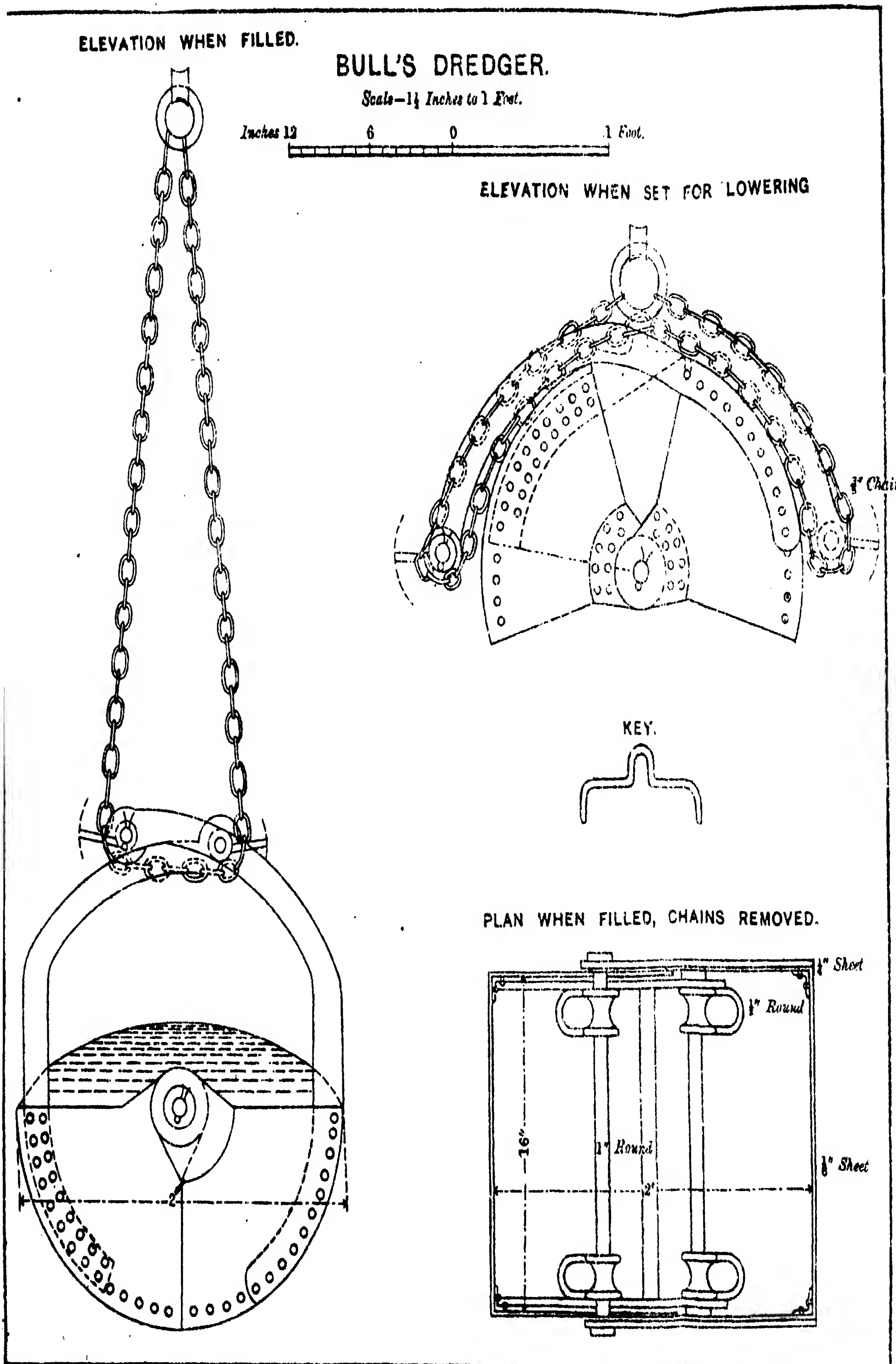


IRON PONTOON BRIDGE AT AGRA 23 ROADWAY.

Scale—10 Feet = 1 Inch.



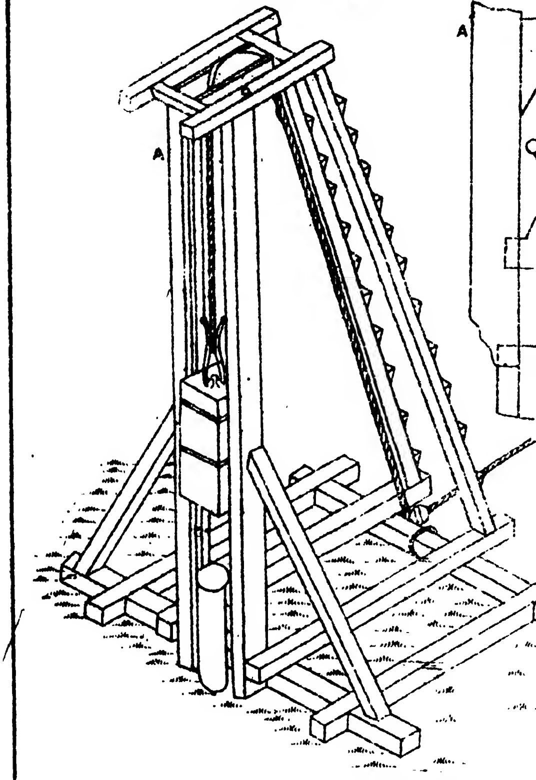
4, S.P.S. U.R.(R) - 1250.



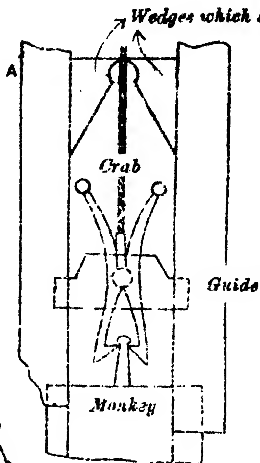
P.S.U.P.(R) 4 S.P.S.,1947,1250, Photo. Zinco., January,

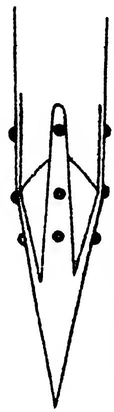
PILE DRIVERS AND PILE.

CRAB ENGINE Fig.



Scale $\frac{1}{100}$ of full size.





PILE WITH ORDINARY SHOE.

NASMYTH'S PILE-DRIVER.

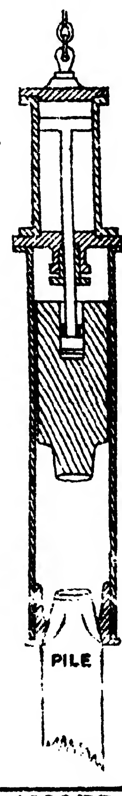
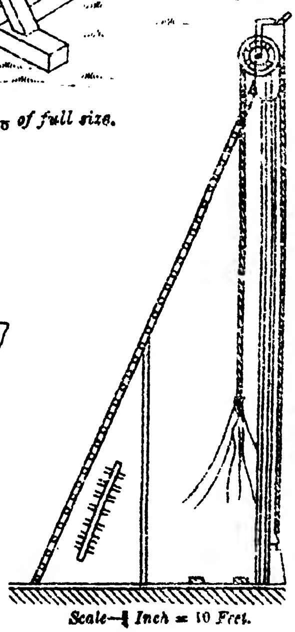
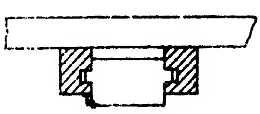


Fig.
RINGING ENGINE.



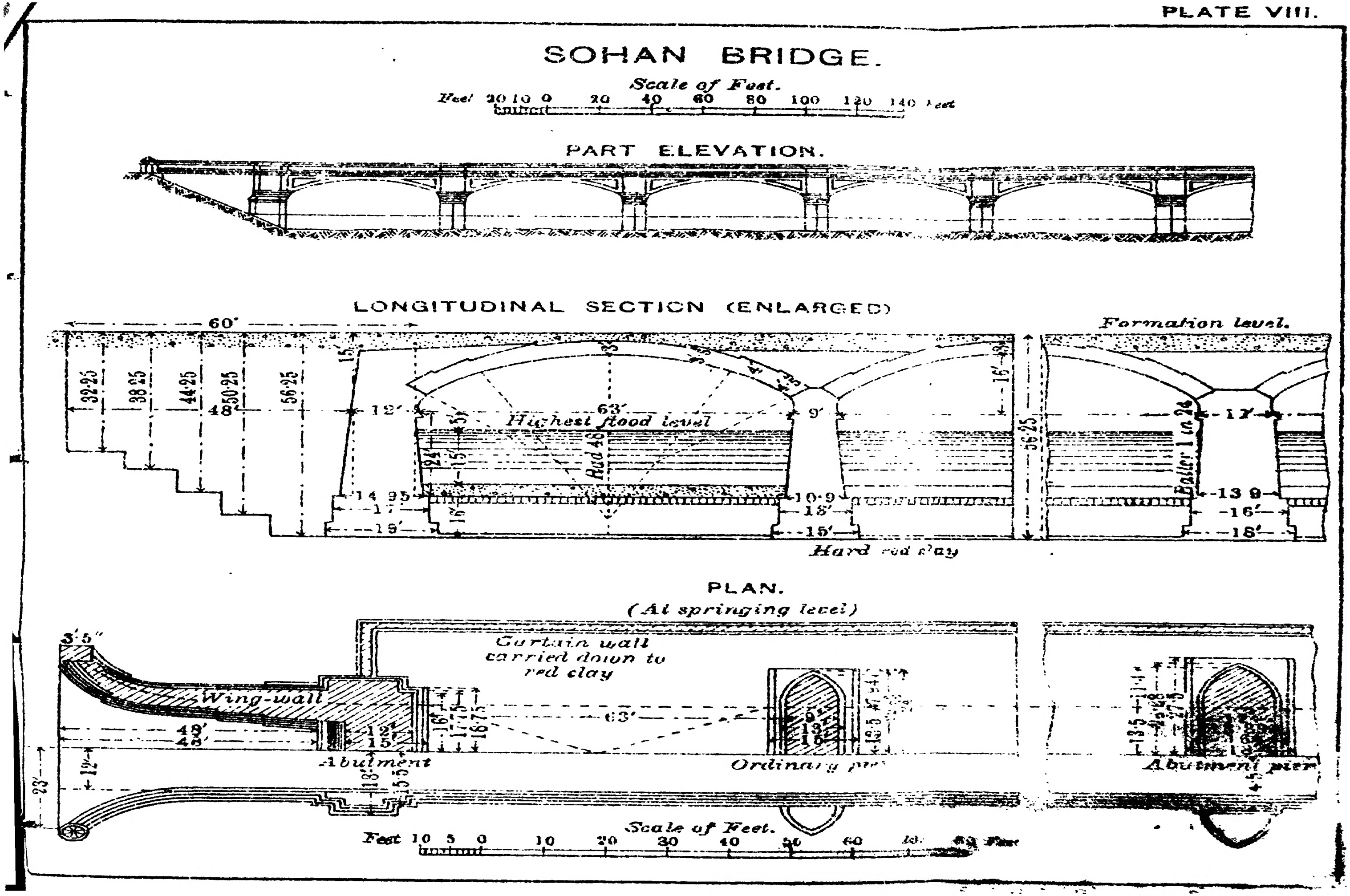
Scale—$\frac{1}{8}$ Inch = 10 Feet.



PLAN OF MONKEY.

P.S.U.P.(R) 4,S.P.S.-1547-1950.

PLATE VIII.

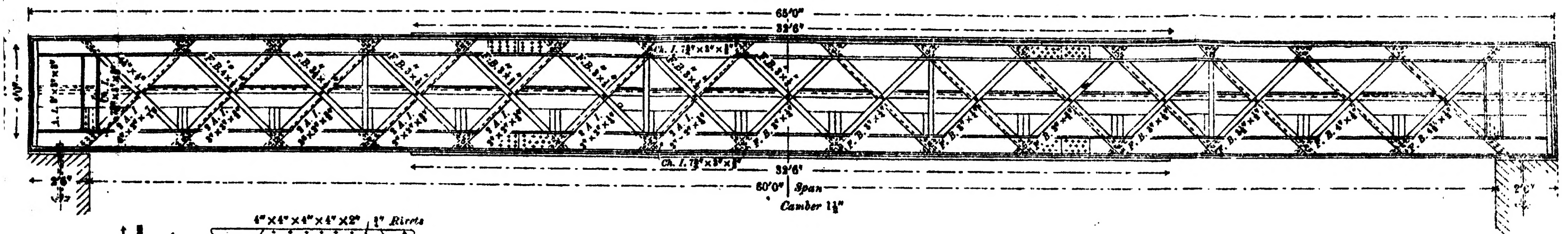

SOHAN BRIDGE.
Scale of Feet.
PART ELEVATION.
LONGITUDINAL SECTION (ENLARGED)
Formation level.
Highest flood level
Rad 48
Batter 1 in 24
Hard red clay
PLAN.
(At springing level)
Curtain wall carried down to red clay
Wing-wall
Abutment
Ordinary pier
Abutment pier
Scale of Feet.

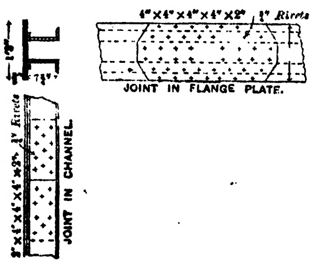
LATTICE GIRDER BRIDGE.

60 FT. SPAN 12 FT. ROADWAY.

Scale—1 Inch = 5 Feet.

ELEVATION.

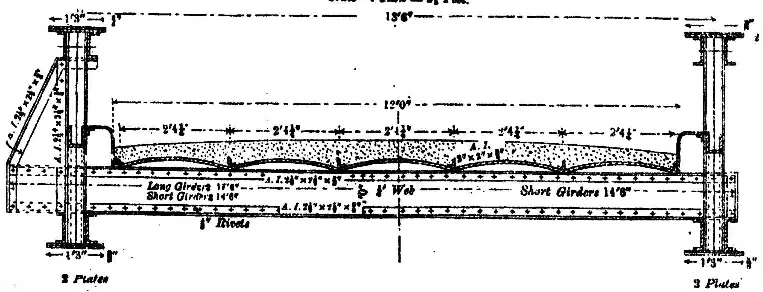


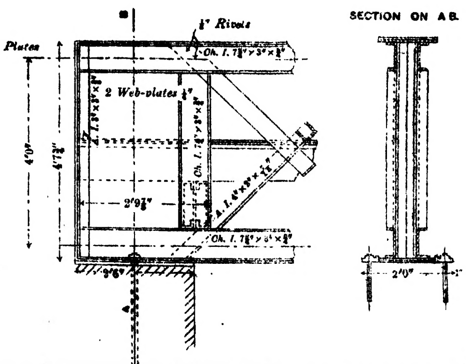


HALF CROSS SECTION.

THROUGH LONG CROSS GIRDERS. THROUGH SHORT CROSS GIRDERS.

Scale—1 Inch = 2½ Feet.





QUANTITIES IN ONE SPAN.

Description.	Tons.	Cwts.	Qrs.	Lbs.
Lattice Girders, ...	10	15	3	0
Cross Girders, ...	3	18	3	0
Flooring, ...	5	8	0	0
Wall-plates and Bolts, ...	0	9	1	0
Total in one Span	20	11	3	0

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