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The foundations of important structures, though unseen or unnoticed by the public, may involve some of the most difficult planning and construction work of the entire project. Here in the metropolitan section of New Jersey, are shown the Pulaski Skyway and other bridges, large industrial structures and important buildings whose safe founding has involved engineering of a high order. ***(Courtesy of the New Jersey State Highway Department.)***

FOUNDATIONS OF STRUCTURES

by

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FOUNDATIONS OF STRUCTURES

According to the grace of God which is given unto me, as a wise masterbuilder, I have laid the foundation, and another buildeth thereon . . .

—I CORINTHIANS 3 : 10

PREFACE

The planning of foundations for structures is more of an art than a science. The conditions at any particular site must be taken as they are —the result of the processes of nature in the formation and disposition of soils and rocks, often modified to some extent by the works of man. These natural materials differ vastly in their physical properties. They may be mixtures that possess to some extent the various characteristics of their constituents, and erosion may have assembled them in an almost infinite number of combinations and stratifications.

In foundation work the engineer must use rocks and soil as structural materials. He realizes that the planning of the superstructure should be based upon the characteristics of the materials of which it is to be built, and he may select and use the materials that will best serve his purpose. On the other hand, in planning the substructure, he must use whatever soils are already there, or he must devise ways to improve the situation in the interests of safety and suitable economy.

Although it is practically impossible to set up rules, regulations, and detailed recommendations for the solution of all the problems that arise in the planning of foundations, there are many basic principles that may be helpful in such matters. In this book the author has endeavored to present some of these principles and to show their application in specific cases. In a field such as this, a person will inevitably have ideas that do not agree completely with those of everyone else. Nevertheless, the author believes that lessons learned through his own experiences and those of his friends will assist the reader to develop engineering ability that he can use in solving the foundation problems that may lie before him.

The author has not attempted to write a book on soil mechanics. He has endeavored to show how to take soils as materials and build substructures upon them. The types illustrated are mostly those that are encountered in ordinary construction. Major and complicated foundations are usually planned and designed by men of long experience who do not need to read this book. The young engineers and those of limited experience are the ones whom the author wishes to help particularly. The successful performance of their structures is of vast importance to them and to those whom they serve.

The curricula of our engineering schools are already so crowded that many important subjects can be studied in only a brief introductory manner. The author has planned this book so that after the first four chapters the reader may study the first few articles in each of the remaining chapters and from them obtain the general principles involved and the methods of attack that may be used.

He who is in engineering practice may wish to study in detail whatever else seems to offer assistance. For this reason many of the problems are worked out numerically so that he can be sure, when studying alone, that he understands the engineering and computations involved. All calculations were made by the use of the slide rule, and many of the results rounded off to two significant figures.

In the detailed designing and analysis the author uses approximations that he believes are easy to understand and apply. It is believed also that the methods used yield safe, reasonable, and satisfactorily economical results. When one considers the broad assumptions that must be made in the selection of allowable bearing pressures, applied loads, estimations of the characteristics of soils, and the action of relatively thick members of reinforced concrete, he will realize that subsequent calculations need not be made with a refinement that is utterly inconsistent with the accuracy of the data upon which they are based. However, the approximations should lean toward the side of safety. The great economies or expenditures in foundation engineering are "made" at the time that the basic plan is determined. Refinements of the computations thereafter affect the cost less than one might suppose.

In practice one should learn all that he can, within reason, about the conditions at a given site. He should then determine what types of foundation are practicable in this specific case. Then he should compare their cost, suitability, ease of construction, and safety. Finally, he should adopt the foundation which, in his judgment, is the best. After that, he may proceed with the detailed parts of the design. Repeatedly, the author tries to show the reader how to size up a situation and how to tell which of several alternatives is the best.

The author wishes to thank all who have contributed photographs, drawings, and data for his use. They have been very helpful. He is especially grateful to Ayres C. Seaman who read the manuscript, and to L. A. Warner for other assistance.

CLARENCE W. DUNHAM

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June, 1950

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1

INTRODUCTION

1-1. Importance of foundations. It is obvious that the foundation must be adequate if a structure is to be safe and satisfactory. Though the foundation is inconspicuous and may be unnoticed by the public, the planning of its basic features and the design of its parts may involve some of the greatest engineering skill and the best judgment in connection with a project. The construction of the foundation may require some of the most difficult work of all the operations carried on in the field.

It has been said that he who designs and builds important and difficult foundations does not sleep well at night. This may be an exaggeration, but it is not without a bit of justification. If a steel girder is too weak and flexible, it usually can be strengthened; if a column is found to be inadequate, it may be reinforced or assisted by intermediate columns that were not at first intended; but, if the foundation yields unevenly and ruptures a structure, if settlement causes the building or pier to tilt badly, if the subsidence is so much that the structure is not satisfactory and usable, then little can be done to improve the situation. One can seldom rebuild an inadequate foundation and repair the edifice without excessive expense. Although the Leaning Tower of Pisa is known throughout the world and even the objective of many a traveler, the modern engineer should not expect that one of his structures that settles unevenly will compete for fame with that edifice.

The planner and designer of the foundation must assume the responsibility for its performance. The construction men may have great difficulties in carrying out the intent of the plans, but they should not be held responsible for unwise judgment that may have been used in the concept and design of the project. Yet those who plan the structure and who make the vital decisions are faced with problems beset by uncertainties. The vagaries of nature have prepared the materials and placed them as they are. The characteristics and probable actions of these materials under loads may be difficult to ascertain; floods may wash away the soil or inundate the structure; moving ice and storms may exert pressures of unknown magnitudes; and frost may cause heaving and subsidence. Even the works of man may aggravate the difficulties where a sunken barge or heaps of riprap lie buried beneath the mud and sand, where some old sewer cuts across the site far below the surface, or where part of the terrain consists of questionable fill such as old mattresses and bedsprings. Even the prospective owners often seem to be determined to build the most important structures in the most inopportune places—and for exceedingly meager sums of money.

Is the engineer to be cowed by these difficulties? Is he to go home and wait until the difficulties disappear? It is up to him to obtain all the information possible about the problems confronting him, to determine what courses of action are open to him, to study various alternates that might be used to support the structure, to visualize the probable action of those alternates, to estimate their approximate costs, to decide upon the relative feasibility of their construction, to recommend that which he considers the best, and last, but not least, to explain to his clients the nature of the problems and the reasons for his recommendations.

Thus, the major decisions may rest upon his judgment and engineering sense. These qualities are developed and made reliable through years of study and experience, yet study and experience alone are not guarantees of expert ability if they have not been of the right caliber. One of the greatest of an engineer's assets is the ability to visualize and to think clearly. Another man who does not possess similar ability and understanding should not be criticized too severely because he is unwilling to accede quickly to these decisions that depend so much upon intangible judgment.

1-2. Definitions. What is meant by *planning* and *design?* Hardy Cross has aptly stated that, in substance, planning is the determination of whether or not the project is worth building, what its general proportions are to be, how it will fit into its surroundings, what is required for it to serve its purpose properly, and of what materials it is to be built. Design is the next step in the refinement of the plan. It is the determination of the framing scheme, the choice of the positions and types of members, the obtaining of a scale upon their sizes to see that the scheme is feasible, the selection of what is to constitute the many related accessories, and the perfection of the layout to see that the structure will serve its purpose well. During the planning and designing, basic decisions are made. If these decisions are unwise, minor refinements and the detailed calculations of sizes and dimensions can do little to remedy the situation. In some cases, the word *design* is used to mean the entire office part of the concept and development of the project in contrast to the construction work in the field.

The word *foundation* may denote (1) the soil or rock *in situ* to which are transmitted the forces caused by the dead load of the structure and by all other loads applied to the latter, (2) the total ensemble of the structural parts of the substructure that serves as the medium through which the weight of the superstructure and the forces due to the loads upon it are transmitted to the supporting soil or rock, or (3) the combination of both of these. The sense of the text will make the meaning clear in most cases; Fig. 1-1 will help to clarify the meanings.

The line of demarcation between the superstructure and the substructure may not be determined clearly. For convenience one may define these parts as follows, referring to Fig. 1-1 as an illustration:

1. Superstructure denotes that portion of a structure which is built essentially and directly for the use of man regardless of how the structure is supported. For example, in Sketch (*a*), the owner desires to have a build-

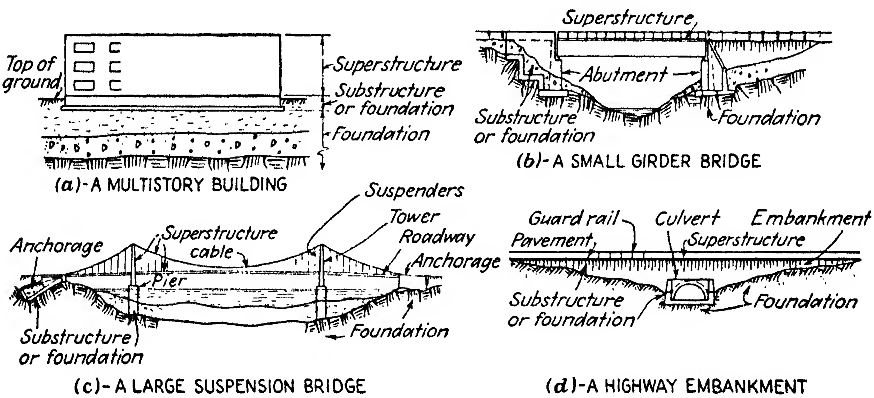


FIG. 1-1. Illustrations of superstructures, substructures, and foundations.

ing of given size and for specific uses; in (*b*) and (*c*) he wants a bridge to hold him up while he crosses a stream; in (*d*) he wishes to have a roadway to carry the public's cars yet to permit the stream to continue in its course. In general, these superstructures are what they are because of the uses for which they are intended, although local conditions of many kinds may temper one's desires regarding what he wants if he is to attain his objectives with safety and economy.

2. Substructure denotes the portion of a man-made structure that is needed to hold the superstructure in place and to transmit all forces due to the superstructure and its use to whatever the supporting material may be. Thus, in Sketch (*a*), the footings and foundation walls spread the forces to the soil; in (*b*), the abutments transmit the reactions of the bridge to the rock; in (*c*), the piers and the anchorages hold the bridge in place even though the anchorages merely resist sliding and the upward pull of the cables; in (*d*), the embankment and the box culvert with its footings transmit the loads to the gravel. In Sketch (*d*), the pavement may be looked upon as the superstructure; in the case of an earth dam, the earthwork itself is the superstructure and needs no separate or extra parts to bear upon the soil. A telephone pole set in the ground has no separate substructure although its embedded portion serves the purpose. The fine distinctions between the superstructure and the substructure are not important; of vast importance, however, is a clear understanding of how to plan each structure, of how to build it, of how it will function, and of how the foundation will behave.

1-3. Purpose. It is the purpose of this book to help the reader understand the character of some foundation problems, to show him how to devise possible solutions for such problems, to enable him to improve his ability to determine the advantages and disadvantages of these solutions, and to help him develop his engineering judgment so that he can make his decisions wisely. Therefore, the endeavor is to illustrate principles and structural action, to show various possible solutions for specific problems, and to indicate how an engineer might compare these solutions in order to make his decision. In many cases of actual work, the schemes that were rejected and the reasons for such action are more instructive than a mere examination of the adopted design.

Many types of foundation are to be studied. Ordinary footings for columns and walls are built in vast quantity. Some conditions require the use of heavy mats; others require piles, deep piers, or even floating foundations. Cofferdams and deep caissons may be needed, too. All these are to be discussed.

Much depends upon whether a structure is large or small, heavy or light, rigid or flexible. Thus, its requirements become the starting point in the planning of foundations. When the soil conditions are exceedingly bad and when it will be too difficult to support the structure, a revision of the plans for the structure itself may be necessary.

Foundation problems vary so greatly that one cannot illustrate all of them adequately. Nevertheless, the general principles of the action of materials and structural elements, when properly understood, are tools that may be used in spite of the variation of the details of the problems to which they are applied. The substructure and the soils around and beneath it are all a part of the same general problem; the latter should be studied as carefully as the former.

Real economy may be difficult to ascertain. For example, at a power plant located at tidewater, the boiler feed pumps were founded on the roof of a reinforced-concrete intake tunnel. Salt water had flowed through the tunnel for something like 20 years. A short time ago the roof gave way, and the pumps dropped into the water. An investigation showed that the bottom rods of the roof had apparently rusted in the salt water and salty air so as to spall off the concrete cover below them. They had then continued to corrode until they were fatally weakened, at which time some gave way. The transfer of load to neighboring weakened rods was more than they could stand, so they failed also, and entire collapse followed. The vibration of the pumps may have aggravated the difficulties, but this is uncertain. At any rate, what seemed like an economical arrangement—the use of the tunnel as a pump foundation—proved to be expensive in the long run. One should always consider the distant future when he plans the foundations for equipment upon which the operation of a plant depends.

1-4. Economics. As D. P. Krynine has said, "When designing foundations, the engineer should consider three signs; *i.e.*, +, −, and $." The last of these is to be borne in mind always, and there is much to consider besides numerical calculations. How much more should one spend in order to reduce a structure's settlement by 25 per cent? How much more expense is justified in making sure that there will be no appreciable settlement? With the money available, what type of foundation will serve one's purpose the best? Such questions as these are important; good answers to them are often difficult to find, yet they must be sought; and the engineer must face the consequences of his decisions because he has no opportunity to bury his mistakes.

Perhaps a building costing $1,000,000 is to be built upon weak plastic soil. Will the soil support it safely and satisfactorily? Should $100,000 more be spent to provide piles that will transmit the loads to firm strata below the questionable soil? Slide rules and calculating machines do not provide the answers. Solutions must be based upon engineering judgment, judgment that is developed through years of experience and study, judgment that assists the engineer in weighing the evidence, in predicting the probabilities, and in making a decision. Such problems are not the place to exercise blind opinion, to resort wholly to precedent, and to do comforting wishful thinking. It is no wonder that Hardy Cross has often said that civil engineering is an art, not a science.

Almost any reasonable structure can be built and supported safely if there is enough money to spend. This, however, is seldom the engineer's problem. In general, he must do the best he can with limited funds, or he must accomplish safe results with a minimum of expenditure.

The first of these—basing one's decisions upon the best that can be done for a stated number of dollars—does not necessarily produce results that constitute good foundation engineering. The engineer is entrusted with the safety of life and property. When a structure fails and someone is injured, nobody will consider any shaving of costs to have been justified; when a structure settles or tilts so as to impair its value and usefulness, the owner will not believe that he was to blame because he set unreasonable financial limitations.

If an engineer is asked to plan and design a foundation for a specific structure and to keep the cost within a stated limit, it is his duty to use the utmost skill of which he is master to devise a good safe economical solution of the problem. On the other hand, if no satisfactory design can be made for the money available, it is his duty to see that his client realizes the situation and the dangers involved. The engineer should, if necessary, refuse to go ahead with a project when he is convinced that it is unsafe or unwise, in spite of the pressure that will surely be put upon him. He is sometimes the prospective owner himself; in other cases he is the owner's adviser and, as such, should give his honest considered opinion regardless of whether it is pleasing. His position is similar to that of a physician who may have to tell his client that an operation is necessary even though such news is exceedingly unwelcome.

The planning and design of a safe foundation for a minimum expenditure form the usual type of problem confronting the engineer. He is always and inevitably concerned with economics. There is an old saying that an engineer is one who can build for a dollar what anybody could build for two. Regardless of economic pressure or of its absence, a good engineer will strive to make the utmost use of men, materials, and money. This is engineering!

2

SOILS AS FOUNDATION MATERIALS

2-1. Introduction. It is assumed that the reader has previously made at least a slight study of soil mechanics. Therefore, the details of the analyses and experiments that are customarily made in a soil-mechanics laboratory are not discussed. Nevertheless it is desirable to give a brief description of the materials that one is likely to encounter in the field, and to provide a condensed discussion of their properties and of their probable action when used for the support of a structure.

The use of soils as foundation materials and for various other works of man dates from a time far beyond the dawn of his recorded history. The knowledge of soils that was obviously possessed and used by the engineers and builders who preceded us was far greater than we sometimes admit. If one stops to think, he will recognize that these men were exceedingly keen and capable, and they often accomplished excellent results in spite of the limited techniques and equipment with which they had to work.

The modern engineer who engages in foundation work has great need for a thorough understanding of soils as structural materials. He will need a keen sensibility to detect danger when it exists. He will need the ability to obtain and evaluate evidence upon which to predict the probable action of soils in innumerable varieties of arrangement and properties. The tests made in a laboratory may be exceedingly helpful, but observation in the field, a knowledge of the performance of structures upon comparable materials, and the proper use of imagination are helpful, too.

2-2. Types of foundation materials. For convenience, foundation materials have been grouped into a series of types. The properties of those in any one group may vary considerably because of different compaction, moisture content, and physical make-up; nevertheless, a knowledge of the properties and behavior of one member of a group or type will assist a person in anticipating what to expect of other similar soils.

The definitions of the groups as used herein and a brief description of each type of material are as follows:

1. Bedrock, or *ledge rock*, is sound hard undisturbed rock in its native location, of indefinitely great extent, not broken up by harmful seams and cracks, and underlain by no material except rock. The geological type of material composing bedrock may be such materials as igneous, sedimentary, and metamorphous rocks, conglomerates, and even slates and shales, although the reliability of these last two may be questionable. The excavation of bedrock cannot be accomplished by hand or mechanical shovels but requires blasting.

2. Shattered or *broken rock* is assumed to be an intermediate stage between bedrock and soil (earth material). It is usually in its natural position above or alongside sound bedrock. It is likely to have seams that are filled with rock fragments or even with claylike materials. Hard pieces may be separated completely by layers of soil, and erosion may have caused much of the fine material to be carried away.

3. Boulders are fragments or pieces of rock that have been broken away from the bedrock. They may or may not have been transported far from their original positions relative to the "parent" bedrock, but they are usually considered as pieces that have been moved somewhat and worn by water or grinding. For descriptive purposes, boulders will be limited to pieces that exceed 3 in. in maximum dimension.

4. Gravel is composed of pieces of rock that are from 3 in. to 2 mm. in size. In ordinary cases, gravels are composed of particles of many sizes of more or less rounded pieces, and they generally contain some sand. A mixture of gravel and sand is generally classed as gravel. Crushed stone and naturally angular material may be called *stone*. *Pea gravel* is a name that is sometimes used to denote a gravel having particles from 2 mm. to approximately ¼ in. in size.

5. Sand is composed of small rounded or angular particles of rock that vary in size from 2 to 0.02 or 0.05 mm., depending upon what system of size classification is used. The following subdivisions of the broad classification are often found to be useful when describing sands:

Fine sand:	0.05–0.2 mm.
Medium sand:	0.2–0.6 mm.
Coarse sand:	0.6–2.0 mm.

6. Silt is composed of fine particles of rock that range in size from approximately 0.05 to 0.005 mm. *Rock flour* is another term used to denote a material composed of very fine particles of rock. *Organic silt* is silt mixed with organic matter and has a bad odor. It is often found in river beds, lake bottoms, and deltas.

7. Clay, or *clay soil*, is composed of exceedingly fine particles of inorganic material, less than 0.005 mm. in diameter. These particles are assumed to be scalelike, and they are generally poorly consolidated. A *varved clay* has noticeable thin strata that are formed by variations in sedimentation during each season or year.

8. Hardpan is generally a gravel and sand mixture with enough clay or silt so that, under terrific pressures such as those caused by glaciers, the mixture has attained a high density, and the particles are partly cemented together.

9. Caliche generally denotes gravels, sands, and silts that are at least partly cemented together by "desert salts," such as calcium carbonate, that are carried into the soil in solution and then crystallized because of the evaporation of the moisture.

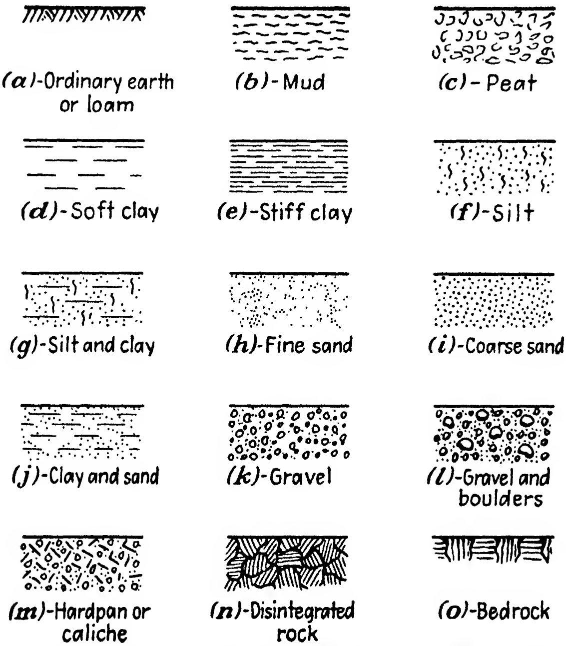


FIG. 2-1. Examples of probable symbols to represent various foundation materials.

10. Loam is a mixture of sand, silt, or clay, or a combination of any of these, with some organic matter—*humus*—and is the soil that is of great value to agriculture. It is sometimes called *topsoil* in contrast to the *subsoils* that contain little or no organic matter.

11. Adobe is a heavy-textured alluvial clay, often associated with desert regions of the Southwest.

12. Gumbo is a very fine claylike material that becomes very greasy when wet. It occurs in some lowlands and central states.

13. Mud is often a slimy, sometimes sticky, mixture of earth materials and water in a fluid or weakly solid state.

14. Peat is partly decayed organic matter, as found in swamps.

15. Muck is generally a mixture of organic matter and inorganic earth materials, although the terms *muck* and *mud* are often used synonymously, *16. Loess* denotes a wind-blown (aeolian) deposit of very fine sand, formed somewhat as are snowdrifts.

17. Bentonite is a fine-grained volcanic ash, like dust. It swells when it is wetted.

Because of the almost infinite variety of mixtures of soil materials, it is customary to describe some soils by means of adjectives that reveal the nature of the most important secondary material that is mixed with the primary (predominant) material. Such descriptions as *sandy clay*, *clayey sand*, *silty clay*, and *gravelly* (or *stony*) *clay* are useful. Such a term as *sandy loam* denotes a loam that is primarily sand. As a rule, sandy materials are "light," whereas clay materials are "heavy" soils, these terms being descriptive of the use of the soils in cultivation rather than from the standpoint of unit weight.

It is desirable to picture soils by symbols when studying, designing, and illustrating foundation problems. There seems to be no set standard of symbols; each office may have a system of its own, or its men may invent the picturization for a specific case. In order to secure some reasonable consistency in the drawings herein, an attempt has been made to use the symbols shown in Fig. 2-1; additional ones will be used as necessary.

2-3. Origin of soils. Soils are chiefly the products of one or more of the following:

1. Physical (mechanical) disintegration of rocks due to erosion, wear, and crushing
2. Chemical decomposition of rocks and rock materials; *e.g.*, oxidation and hydration
3. The mixing of the products of vegetation and other life with inorganic materials

The combination of mechanical and chemical actions that produce soil is often called *weathering*. *Leaching* is the process of removing from soils the water-soluble materials therein by the action of rainfall or underground flow.

Long-continued and complicated have been the geological processes of the breaking up of igneous rocks, the transportation and deposition of the resultant materials, the formation of sedimentary rocks by means of pressure and heat, the breaking up of these last rocks again, and perhaps many cycles of such changes. The materials may have been transported many times and through long distances by water, wind, gravity, and glacier; even man may have contributed to this movement. The soil structure—grading and arrangement of the particles—may affect the properties of a soil composed of a given material, and so may the physical pressure that is or has been applied to the soil. The presence or absence of water, too, may affect greatly the physical properties of these soils as foundation materials. Is it any wonder then that soils are found to be so varied in their character and behavior?

One may derive considerable benefit by visualizing the probable manner in which the soils that he must utilize were formed. Some soils—even thick strata—may have been formed *in situ* by the weathering of the rocks through long periods of time. Such a case is pictured in Fig. 2-2. Such an arrangement may occur in level and slightly rolling territory in the South. In general, however, the force of gravity causes pieces that are broken from bedrock to fall and form slopes of talus, as pictured in Fig. 2-3(a); water carves ravines through weak spots and crevices, as pictured in Fig. 2-3(i), and carries the particles far away; running water continually carries particles from higher to lower elevations and tends to flatten out the slopes and

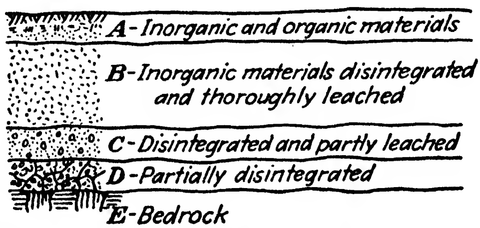


FIG. 2-2. General character and distribution of soils formed by disintegration in place.

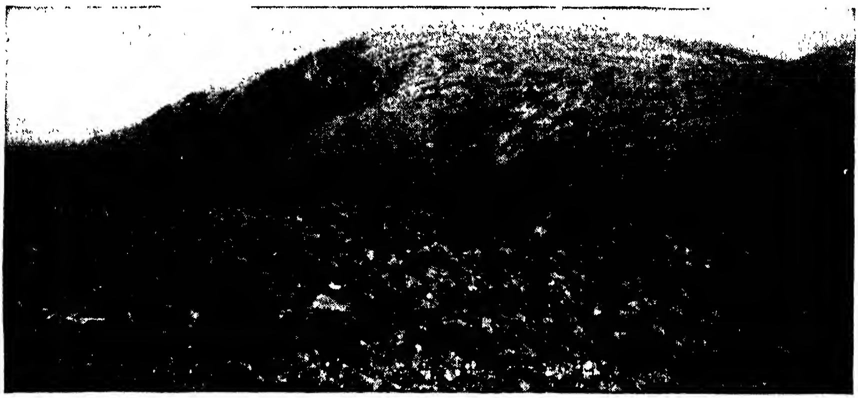
cause the sediment to settle elsewhere, as shown in Fig. 2-3(c); the wind removes the fine particles as fast as weathering produces them, as exemplified by the rocks in Fig. 2-3(b), forms sand dunes, and even "cuts" many rocks into strange shapes; and sometimes volcanoes spread lava and ashes over vast areas.

The glaciers that once covered the northern portion of the United States scraped away a great deal of the soil, ground off much of the disintegrated rock of that region, and deposited boulders, gravel, sand, and clay over wide areas. Long Island, N.Y., is believed to be terminal moraine left near where the glacier's face met the sea, where much of the ice melted and dropped the imprisoned materials, or where it ceased to push and carry them along. Except for glacial clays, these glacial deposits are usually neither stratified nor homogeneous. Even in short distances great differences are likely to occur in the "overburden" or soil covering the bedrock of glaciated regions.

Beach and shore deposits are formed by the action of the waves and tides of the ocean and by the waves of large lakes. These forces of nature cut away the rocks, grind up the pieces, and sometimes carry the sands far along the coast.

Flowing water has surprising ability to transport materials. One who has a garden on a hillside can see its effects after a heavy thunderstorm. A

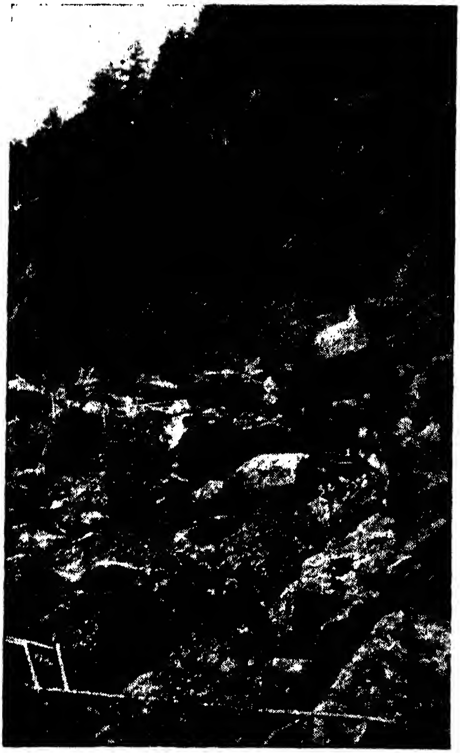
(a) Picket's Post near Phoenix, Ariz., showing talus slopes around base of mountain. A "jumping" cactus is shown in front of the large greasewood bush in the foreground.



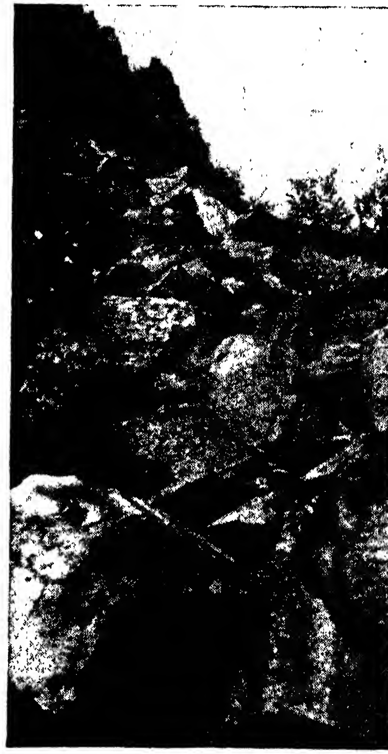
(b) Mt. Washington, New Hampshire, showing the top of Tuckerman's Gulch of skiing fame. Notice how the wind has removed all fine soil.

(c) A swampy backwater in the Adirondack Mountains of New York. The lily pads show the presence of shallow water. Sedimentation fills up such places relatively quickly.

FIG. 2-3. Illustrations of nature's products and processes.



(*d*) Polar Caves in New Hampshire. This mass of boulders was caused mostly by glacial pressure that broke them off the cliff at the right.

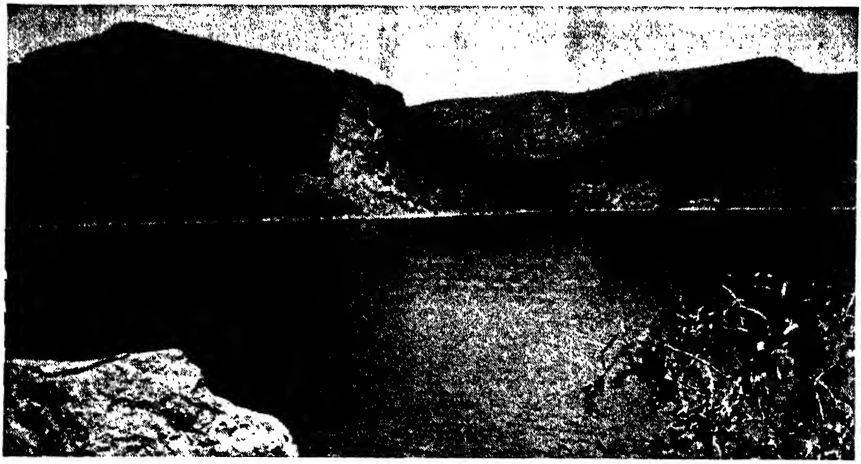


(*e*) The Sleeping Giant, in Connecticut, a trap-rock dike. Erosion has caused large accumulations of talus.

FIG. 2-3. (*Continued*)

fisherman who wades in a deep fast river that has a slippery rocky bottom will realize this, too. The Grand Canyon of the Colorado, the Palisades of the Hudson, the Niagara Gorge, and the Delaware Water Gap are spectacular examples of the cutting action of running water; however, the myriad small ravines, washes, and streams of the country are more important in their total effects upon the removal, building, and transporting of soils.

When a stream flows into a lake or other relatively still water, sedimentation occurs. This action is pictured in Fig. 2-4. Assume that stream A flows down a rocky bed and then empties into a pond at B. The velocity of flow is reduced quickly so that at C it is slight, at D and E it may be negligible. The solid materials that are carried in suspension or rolled along the bottom, especially in time of flood, are discharged into the pond. The heavier particles like gravel will come to rest first, the coarse sand next, then the fine sands and silts, and finally the clay which, being in suspension and settling slowly, may be transported far into the pond. Sedimentation in this manner is a classifying process. However, in times of small flow,



(*f*) Canyon Lake on the Salt River in Arizona. Notice the tilted, stratified rock cap at the left and the wedge of light-colored rock that has filled a great crack.

(*g*) Nigger Head at Douglas, Ariz., protruding through a great valley containing perhaps 1,800 ft. of various strata of clays, sands, and gravels.

(*h*) The Superstition Mountains in Arizona and the "floor" of the "desert." Caliche is often formed in such territory.

FIG. 2-3. (*Continued*)

(i) The Flume in the White Mountains of New England. This shows how a small mountain stream has carved a deep ravine through bedrock.

(j) A marshy flat along the Connecticut shore of Long Island Sound. Such deep accumulations of sand, clay, and muck are likely to be formed by the combined action of sedimentation and shore erosion.

FIG. 2-3. (Concluded)

the fine materials will settle close to B; then they may be scoured out again and carried farther on during the next freshet.

The ground at a particular site may be composed of many strata of varying character. As an example, consider the situation pictured in Fig. 2-3(g). This broad level plain of semidesert would seem to offer no serious problems in building foundations, yet plenty of them arose in connection with the design of the foundation for a large stack, as described in Chap. 16. According to verbal reports given to the author, wells driven in this valley have penetrated 1,500 to 1,800 ft. of clay, silt, sand, and gravel strata without encountering rock. The borings taken at the site of the stack revealed a

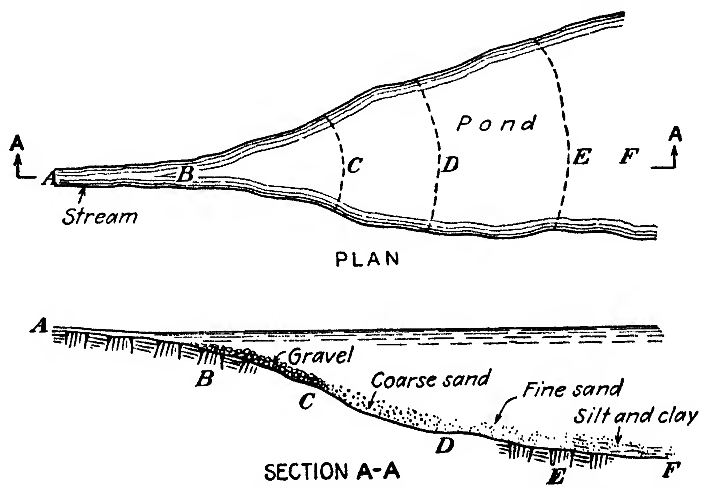


FIG. 2-4. Illustration of sedimentation caused by a stream carrying materials into a pond.

buried watercourse filled with adobe, then a layer of gravel and sand, then strata of clays, more small gravel, a stratum of soft puttylike material that seemed to be volcanic ash, and finally more clay. Lenses of gravel seem to be present, apparently caused by cloudbursts that scoured materials out of one place and deposited them in another.

In other places, the soil may be fairly uniform over large areas. Near Chicago, for example, there is a deep stratum of clay that was deposited when the entire region was under water. At Los Angeles, the flood plain of the Los Angeles River and the Rio Hondo contains a deep layer of fine sand that extends over a considerable area.

Sediments may accumulate far more quickly than one would think. In a rolling, farm and wooded area of Connecticut, a pond about 100 ft. wide, 250 ft. long, and 3 ft. deep was made to form a skating rink for a boys' school. One would not have suspected that a babbling brook would carry much solid material, yet the pond filled up so badly in approximately 15 years that only a few inches of water remained, and the ice would freeze solidly to the bottom.

These matters of erosion and the formation of soils are only a few illustrations of the workings of the laws of nature. They are given to show the reader that it may be very helpful to him if he will try to ascertain and visualize the geological history of the site upon which he is to build a structure involving the safety of life and property, as well as the future of his own professional reputation. He may thus learn what to expect, and determine the probable presence or absence of danger.

When an engineer recognizes or suspects such dangerous conditions that the assistance of a capable geologist is advisable, he should not fail to call for help. Doing so is not an evidence of lack of ability on his part. Many millions of dollars may be at stake. The incompetent engineer is he who fails to perceive the dangers when they exist, or "shuts his eyes and takes a chance" rather than utilize the abilities of the best talent available.

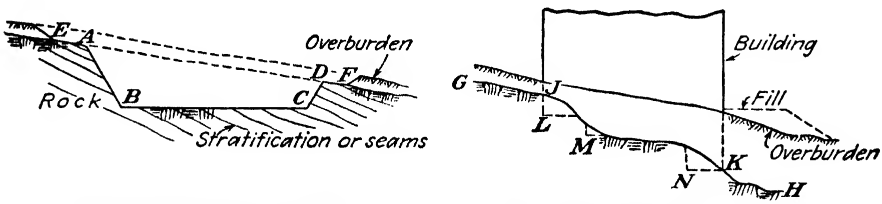
2-4. Some properties of specific foundation materials. The soils encountered in the field cannot always be classified completely in accordance with the definitions stated in books. This must be remembered always. Nevertheless it is proper for an engineer to wish to have available some information as to what he may expect of rather typical materials, and of the loads that can be placed upon them safely. The data given here are for guidance only; they are not given as a haven of refuge behind which the reader can find comfort.

The following are descriptions of the properties of foundation materials in so far as they affect the planning and design of foundations:

1. Bedrock is usually capable of withstanding any pressure that can be applied by man's artificial rock, portland-cement concrete. However, it is important to ascertain the kind of rock, the presence of cracks, the dip of the strata, and the slope of the surface. There may be cases in which the quality of the bedrock itself should be investigated for chemical and physical stability. An extreme case of this arose in connection with the selection of the site for an industrial plant in northern Mexico. The white chalklike volcanic rock underlying a thin siliceous rock cap on a low broad hogback hill was tested crudely by putting small pieces in a shovel containing a little water. The "rock" audibly sucked up the water and disintegrated so fast that it became mushy in a minute or two. This led to the search for another location.

This extreme case illustrates emphatically another basic principle for the engineer to remember; *i.e.*, the hills are hills because their material was more resistant to erosion than were those of the adjacent valleys and ravines. Therefore, there is the possibility that, if dangerous materials are found at the edges of the hill, the softer materials may underly a harder rock cap on the hill. Is excavation likely to remove the protecting blanket and expose the weak materials?

Shales disintegrate easily; so may slates and sandstones. Limestone as a foundation for a dam may fail because of hydration along leaking cracks. If core borings show large cracks, the rock may be suitable as the support for an ordinary building but not for a dam or a great bridge. If the strata dip sharply toward the excavation, as at *A* in Fig. 2-5(*a*), the side *AB* of

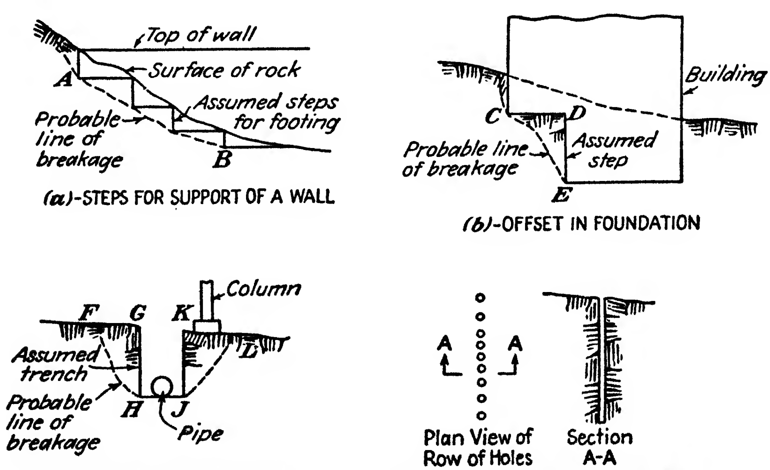


(*a*)-ROCK CUT FOR A HIGHWAY (*b*)-A BUILDING FOUNDED ON ROCK

FIG. 2-5. Excavation of, and foundations on, bedrock.

the rock cut *ABCD* may weather rapidly and cause rock slides. A structure supported on the berm *EA* will be in a relatively dangerous situation compared to one on *DF*. However, if the excavation is a narrow cut running into this hillside, it may be possible for the rock to "arch" horizontally around it. When the foundation is to be on sloping rock, as in Sketch (*b*), benching may be necessary at *L*, *M*, and *N* to avoid sliding of the structure.

The matter of the excavation of bedrock for foundation purposes should be faced realistically. Rock cannot be carved out with a knife; it must be



(*a*)-STEPS FOR SUPPORT OF A WALL

(*b*)-OFFSET IN FOUNDATION

(*c*)-TRENCH FOR DRAIN OR SEWER

(*d*)-ILLUSTRATION OF LINE DRILLING

FIG. 2-6. Excavation of bedrock.

blasted out, and it will break away along its natural planes of weakness—parallel to the stratification or along the cracks. Plans are sometimes made without proper consideration for these practical matters. For examples, refer to the following sketches in Fig. 2-6:

(*a*) Here a retaining wall is to abut into a rocky hillside. Benches are shown cut out of the bedrock. If they are relatively short, as pictured, it is probable that the finished excavation will follow all too closely along the dotted line *AB*.

(*b*) In this case, the rock is shown benched at *CD* in order to save excavation and construction. The rock is likely to break somewhat as shown by the dotted line *CE* so that the hoped-for bench will not exist.

(*c*) This illustrates a drain that is supposed to be near a row of columns in an industrial plant. When the rock is blasted for the trench *GHJK*, the result will probably look more like *FHJL*, and the rock under the columns must be benched farther to avoid sliding of the bases.

(*d*) This pictures the idea of "line drilling" of rock. Holes are drilled very close together so that the rock can be pried off, or so that a plane of weakness is developed with the hope that a blast will break the rock away along that plane but not beyond it. It is needless to say that such work is expensive. When benching is desired along *CD* of Fig. 2-6(*b*), it may be possible to line-drill the face *DE*. However, the blasting performed in excavating down to *CD* is likely to weaken the rock near the corner *D*, causing the latter to crack off. It is better to plan rock excavations with generous dimensions and simple outlines than to attempt to secure sharp reentrant angles as at *C* in Sketch (*b*), corners as at *D*, narrow vertical chases, sharp vertical corners, and undercut pockets.

2. Shattered or *broken rock* is untrustworthy for heavy loads. In general, it should be removed so that the structure may be founded upon the underlying bedrock. Its use as the sides of steep exposed cuts is likely to be dangerous because of weathering and slides. In some cases, pressure grouting with portland cement will seal the open cracks; when the cracks are filled with silt, clay, or very fine disintegrated material, the efficacy of the grouting may be doubtful. Grouting and pointing up of the joints in a badly cracked exposed rock face may be positively harmful because water is more likely to be trapped behind the face where its hydrostatic head and expansion during freezing may aggravate the danger of slides. It is best to found a structure well back from the top of a badly cracked rock slope and to keep well away from the bottom of it.

3. Boulders, especially large and irregular ones, are not reliable for the support of heavy loads because of the weaker underlying and surrounding materials and because of their tendency to tip. Boulders deep down under plastic weak soils in which piles or caissons are to be driven are likely to cause serious difficulties, some of which are pictured in Fig. 2-7. It may be advisable in some cases to remove the large boulders entirely; in others, to found the structure upon the soils above the boulders; or it may be necessary to find a better location.

4. Gravel is an excellent foundation material if it is not underlain by a weak stratum and if it is situated so that scour will not undermine the foundation. Shaley gravel is, of course, an exception. The presence of ground water is not likely to decrease the strength of a well-graded gravel composed of stable materials. Gravels have little capillarity.

5. Sand is ordinarily a good foundation material. However, when it is fine and loose, its strength and bearing value decrease. When very fine and saturated, it may become "quick" and flow out from under even

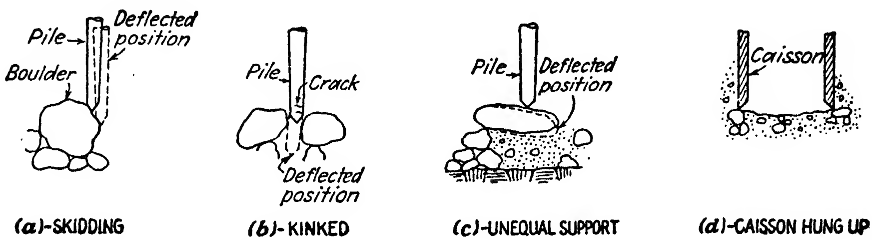


FIG. 2-7. Examples of troubles caused by boulders.

small loads if a way for its escape is provided. Such sand thoroughly trapped, as within a tied steel sheet-pile enclosure, may be trustworthy if one can rely upon the permanence of the restraint. Sites in which quicksand exists should be viewed with great suspicion. On the other hand, damp fine sand may be stronger than the same material when completely dried because of the surface tension of the moisture, as one may realize when he walks on an ocean beach. Medium and coarse sands have low capillarity; when compact and well graded, they have good bearing value; when saturated, they will still hold substantial loads.

6. Silt (nonorganic) is definitely one of the weak unreliable soils with high capillarity. It is not plastic and has little strength when dry. Its presence constitutes a warning to the foundation engineer. Its high capillarity may increase the possibility of damage from frost action. Silt has a low frictional resistance and will flow easily into excavations; even when shoring is used around an excavation, the ground water that leaks in may carry enough silt with it to cause undermining and settlement of adjacent areas. Silts—and fine sands, too—are likely to "blow up" in the bottom of an excavation if a hydrostatic head exists under the bottom. Silt once badly disturbed will have a very low supporting value for a long time. Very fine silt with a little clay—and organic silt—may have considerable impermeability, plasticity, and cohesive strength. In the construction of the Lincoln Tunnel at New York, these qualities of the Hudson

River silt enabled the contractor to use far less pressure of compressed air than was needed to resist the theoretical head of water.

7. Clay usually contains and effectively retains considerable water. When dried, it is generally quite hard, and it shrinks and cracks excessively during drying, only to expand again when its moisture is recovered. In a moist state it is highly plastic and compressible; it may even flow slowly under long-continued pressure. Surface water will soften and erode it easily.

Clays may be described as soft, medium, and stiff, depending upon their moisture content and prior consolidation. In some cases, clay will support considerable load. Its impermeability may be an asset when excavations are made because water cannot enter the excavation rapidly. Its cohesive strength enables it to stand temporarily with steep slopes, and to transmit moderate pressures around a small excavation. Nevertheless, clay used as a foundation material should be studied carefully and utilized with discretion.

8. Hardpan will generally resist relatively large unit pressures. Being formed by glacial pressure, it is evident that it is not underlain by soft plastic soils, but it may have many boulders in and below it. The principal danger to guard against is its softening because of weathering, and its erosion by surface or subsurface water.

9. Caliche, being sand and gravel—possibly with some silt and clay—that are cemented together, is generally very strong as a foundation material. However, future leaching of the soluble cementaceous compounds may restore the soil to its former character. Hence, a steep cut may break down so that a structure resting close to its top edge may be endangered. Structures bearing upon "carved" offsets like *CDE* of Fig. 2-6(*b*) may not always have the intended strength of support beneath them.

10. Loam is not a trustworthy material for heavy foundations. Generally being silty and clayey, it partakes of the qualities of these materials; containing organic matter, too, it is weak and compressible. In most instances, it should be removed completely and not utilized for foundation purposes.

11. Adobe is fairly stiff and highly cohesive when dense and moist, but its value as a foundation material when wet is extremely poor, and wetness should be expected at some time or other.

12. Gumbo, as its name suggests, is notoriously poor for a foundation. If faced with the problem of utilizing it, the engineer should "float" the structure or plan it so that settlement will not be harmful.

13. Mud is useful for the support of very light loads only, even when it is moderately compacted.

14. Peat is practically worthless as a foundation material. Although compacted under a blanket of fill, it should not be trusted underneath important structures, even for highway embankments.

15. Muck is only a slight improvement over peat.

16. Loess may be of some value as a foundation material, but it is likely to be less reliable than a sedimentary deposit of equally fine sand. The

FIG. 2-8. Sketch map showing centers of radiation and area covered in North America during the Pleistocene ice age. *(After Chamberlin and Salisbury. From Legget, "Geology and Engineering," McGraw-Hill Book Company, Inc., New York, 1939.)*

compressive strength and compressibility are likely to vary from place to place and cause unequal settlements.

17. Bentonite, when wet, is so plastic and slippery that even a cat might have trouble keeping its footing. One should be wary about trusting it.

Fortunately, the unsatisfactory materials mentioned are localized and are not very extensive. Along water-front areas and near sluggish streams, conditions are sometimes very unfavorable, and these locations are often the necessary or desirable ones for industrial plants. Each foundation problem is to be studied by itself, and the solution should be tempered by considerable conservatism.

Tables 4-2 to 4-5 give assumed figures for the permissible bearing values and other properties of various soils, and are not to be accepted as the absolute values. They are given as qualitative data—as a general scale on the probable values. Local building codes and qualified persons may state figures that are different; the former particularly should be consulted because they should not be violated willfully. However, codes and specifications are prepared for the *guidance* of the engineer; they do not prevent him from being more conservative if his investigations convince him that he should be so in a particular case.

2-5. Soil-mechanics laboratories. The foundation engineer should not fail to obtain any information that can be secured if it will give him important evidence upon which to make his decisions. He should take advantage of the tests that can be made in a soil-mechanics laboratory. It is admitted that they may not tell him everything nor answer all his questions. Nevertheless, he may obtain valuable evidence of the probable behavior of the soils from the test results, such as the determination of particle sizes and grading, moisture content, plastic limit, liquid limit, plasticity index, density, shearing resistance, coefficient of internal friction, cohesion, consolidation, permeability, and physical and chemical soundness. The consolidation tests for clays and the shear test for all materials are probably the most important to the foundation engineer.

These tests should be made, and the engineer should be able to interpret the results and not hesitate to confer with those who made the tests. The engineers and technicians who make the tests personally are likely to form valuable opinions regarding the probable action of the materials under the existing conditions and under the forces to be imposed. Their opinions should be sought and weighed carefully.

Nevertheless, it is important for an engineer to visit the site and to study the conditions personally. Hardy Cross once said substantially that it is advisable for an engineer to visit the site and surrounding terrain even though he sits on a stump for several hours just looking them over. Of course, this means that it is beneficial if he knows what to look for, if he knows enough to understand what he sees, and if he lets his imagination freely picture all the probable things to come that relate to the job. He should make such a visit before explorations are made, and he should make one or several more after the soils have been tested and as the plans are being developed. Then, as the excavation is carried out, he should watch to see that no surprises are encountered. If unfortunate, unexpected, or dangerous conditions are discovered, it is wiser to face and overcome them immediately than it is to stick obstinately to preconceived plans in order to avoid the expense and embarrassment of revising a design.

3

EXPLORATION OF SITE

3-1. Tentative selection of a site. There are many things to be considered before *the* site for an important structure is selected. In the case of a proposed industrial plant, for example, there are business matters to be settled before the general locale is chosen. Some of these matters are the relation of the proposed plant to such as the following:

1. Present properties and business of the corporation
2. Locations, properties, and facilities of competitors
3. Character and quantity of proposed production, and the possibilities of success
4. Source and availability of raw materials, water, and power
5. Location, character, and capacity of the market
6. Labor supply
7. Transportation facilities and costs
8. Climate and living conditions
9. Space required and cost of real estate
10. Taxes, legal restrictions, and special privileges

After the general location is determined, comes the problem of selecting the specific site. There may be several properties among which one can choose.

A bridge, for example, should be located where it is possible to build such a structure and where it can fulfill its purpose of supporting traffic in the best practicable manner. A large office building, a public edifice, or an apartment house may be rather restricted as to location because of other structures, zoning regulations, available space, transportation facilities, and the dictates of business policy.

A discussion of these fundamental questions is not intended. Assuming that a specific site has been chosen tentatively for good and sufficient reasons, what explorations should be made?

3-2. Importance of exploration. The person who fails to investigate the conditions at a specific site before he purchases is inviting trouble. It may be necessary to secure an option on the property with the agreement to purchase, subject to proof that conditions are satisfactory. It is probable that the prospective purchaser must stand the expense of any explorations. However, if he fails to have them made *before* he buys, he may find that he cannot build as he intended without unduly costly foundations; he may be forced to build lower lighter structures covering more area so that there is insufficient space left on the property for other necessary facilities or for future expansion; or he may be forced to buy another property and dispose of the first one as best he can.

A businessman or engineer would criticize severely anyone who started a business project without first determining that the venture could and probably would be successful. Nevertheless, many a piece of real estate has been purchased for building purposes without proper investigation of what was beneath the surface. The engineers were then forced to make the best they could of the situation. In one case, an industrial concern was on the point of buying a large tract of land when the president of the corporation heeded the advice of his engineer and insisted upon subsurface explorations. These revealed such bad conditions that expensive piling would have been needed under the entire structure and beneath all of its important equipment. Another site was selected, and the owners of the first one admitted that they might as well give the land to the city because they themselves would not think of building upon it, and they could not now sell it to anyone.

Wherever it is practically necessary to locate a structure at a specific place, there should be a practicable exploration of both surface and subsurface conditions. The presence of weak soils may force one to choose a lightweight material for the structure so that the intensity of the applied loads will be small, to use long-span construction with a minimum number of individual foundations because each one extending to a firm support will be so costly, and to adapt the layout of the entire project to the limitations imposed by the foundation conditions. On the other hand, hidden rock may prevent or make the intended excavations very costly. Springs and high ground water may increase the cost of construction, and they may cause water to leak into basements unless the construction is waterproofed. Flood waters may cause tremendous damage unless the worst probable conditions are ascertained and the structure planned to withstand them. The cost of these explorations is slight compared to the possible consequences of a failure to make them.

Interested well-intentioned neighbors, friends, and old residents of the district may give their opinions about surface and subsurface conditions at a site. These statements may have considerable value in giving one a knowledge of past floods, wet areas in the spring, filled areas, adjacent mud flats that are a nuisance in dry seasons, unusually high tides, material encountered when digging or drilling wells or when building roads and installing utilities, and the conditions encountered when adjoining structures were built. Although these data may be given with honest intentions, they should be checked. All too often they are based upon misinformation, hearsay, and general impression; or they may be the opinions of persons who are not really competent to advise in such matters.

An industrial plant consisted of several small old buildings. The company decided to build alongside one of them a moderately large addition of mill type. The company's men told the engineer who was to design the plant that the soil was good sand. In fact, they dug a hole 4 ft. deep to prove it and refused to pay for further exploration. The engineer accepted this information. However, when the excavations for some of the footings were completed, he visited the site, took a 6-ft. piece of water pipe, and had some laborers start driving it down into the ground at the bottom of an excavation. When the pipe was down about 3 ft., the penetration per blow increased. Suddenly, the pipe almost vanished. Further driving of more pipe showed that, after penetrating 3 ft. of sand, no substantial resistance was encountered until a depth of 12 ft. was reached. Of course, the foundations had to be redesigned.

What would have been the situation if the engineer had not discovered the presence of this trapped layer of soft soil? When the building settled, the owner would undoubtedly have blamed the engineer because he had not been convinced that thorough exploration was absolutely necessary. Sometimes an owner insists upon the reduction of construction costs by using cheaper materials, weaker designs, and skimpy foundations. Nevertheless, he is loath to admit that he is responsible for the unsatisfactoriness of the structure. The old advertising slogan, "The remembrance of quality remains long after the price is forgotten," contains an important truth.

Other illustrations might be given to show the many troubles unexpectedly encountered when subterranean conditions were not explored. Two more will be described.

Two small but important industrial buildings were to be located at the edge of a harbor. "Everyone" knew that the site was covered with muck and silt and that it was foolish to waste money exploring the area because the structures would have to be supported by piles anyway, over 300 of them. After about 50 piles had been driven, hard obstructions were encountered 6 to 12 ft. below the surface. These proved to be miscellaneous old slabs and chunks of concrete from some demolition made in the past. Seemingly, they had been dumped from barges when the area was under water and had been covered with the muck and silt. Piles could not be driven through this debris without damaging them. Having already driven 50 piles of a type that could not be pulled and reused, the engineer decided to excavate the area and remove the obstructions. After considerable delay and with a substantial expenditure, safe groups of piles for footings were finally driven. If $1,000 had been spent for exploration, the obstacles might have been discovered, the buildings could have been located elsewhere, and considerable money would have been saved.

In another case, an industrial plant was to be built in a foreign country. Residents of the area said that it was all rocky territory, and it looked so.

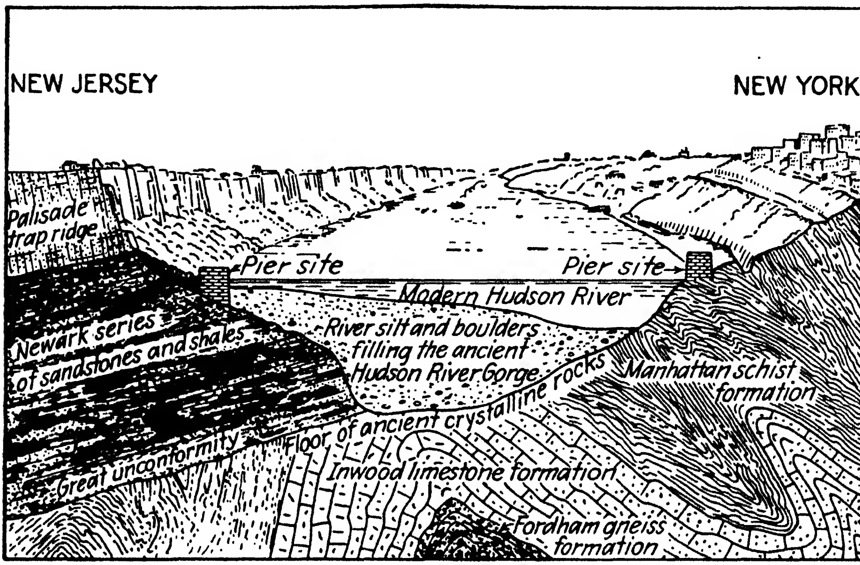


FIG. 3-1. Soil profile at the site of the George Washington Bridge. The vertical scale is exaggerated. The drawing is made with a perspective picture of the adjacent river valley in the background. This is a very effective presentation when used to present data to the general public. ***(Courtesy of the Port of New York Authority.)***

However, photographs indicated deep eroded gullies where some of the structures were to be located. Explorations were finally authorized. The result was the discovery that part of the area was underlain by volcanic materials that softened when exposed to air or water. It became necessary to relocate a large part of this $20,000,000 plant.

Failure to spend the necessary sum for exploration of a site *prior* to planning and building structures upon it is extremely shortsighted. Without this investigation, *almost* everything has been done—but not *all*—to ensure a good job. It is somewhat like the case of the New Jersey man who commuted to New York via the Lackawanna ferry. One day he was late. As the ferry was pulling out, he climbed over the gate at the slip and tried to make a running broad jump onto the deck of the boat. Later, when dragged out of the water, he complained that all that he missed was the last six inches. Yes! The last six inches, but they made all the difference between success and failure!

3-3. Shallow explorations. There are many structures for which shallow explorations are probably sufficient, *e.g.*, pavements, small buildings, retaining walls, and small tanks. Nevertheless, the adequacy of shallow explorations is a matter to be considered carefully.

Driving a pipe or rod a few feet into the ground with a sledge hammer may disclose the depth of muck, peat, or soft silt in a swamp; the thickness of loam or shallow soft clay overlying sand or gravel; and the depth to rock or other firm material when it is near the surface. However, this gives only a vague idea of the qualities of the materials, and the depth of penetration is greatly limited.

By drilling holes with an auger welded to the end of a pipe, one may attain greater penetration into the ground than can be obtained by driving down a rod. The information obtained is still likely to be meager because sandy soils will not stick to the auger so that one can examine them thoroughly, and surrounding sand may run into the holes. Clay will stick too much and clog the auger.

Test pits are more costly, but they enable one to examine the materials in their natural state, to judge their firmness by the resistance to digging, and to see firsthand what the ground-water conditions are. These pits should be at least 2 by 4 ft. in plan so that a man can work in them; if shoring is required, they should be larger. Of course when they are over 6 or 8 ft. deep, excavation is rather costly.

A test pit should not be dug at the location chosen for an important wall or footing if the pit is to be dug below the bottom of the future foundation because the backfill will cause a soft spot. The pits should be located near important points but not so as to endanger them.

If laboratory tests of the soils encountered are desired when digging test pits, reasonably good samples of soil may be secured by forcing a bottomless tin can down full depth into undisturbed soil, excavating around the can, cutting off the earth at the bottom of the can by means of a trowel, and sealing the specimen in paraffin. A chunk of undisturbed clay may be taken by cutting a pair of steps in the clay with a spade, then gouging out a long vertical slot on each side of the sample, and finally cutting off the back and bottom with a wire saw.

3-4. Deep explorations. It is obvious that the exploration of the soil under an important structure should be carried to a sufficient depth to reveal all the information which is needed or which can be obtained practicably to enable the engineer to plan, design, and build the entire structure satisfactorily. What this depth may be in feet cannot be answered by a specific figure for any and all situations. Instead, it should be determined with proper consideration of such matters as these:

1. At least one boring should penetrate a distance equal to the width of the structure unless this exceeds 100 ft.

2. When the average pressure per square foot of the basement or first floor of a building is to be large, the exploration should be carried down far enough to penetrate the soils that may be loaded sufficiently to cause important settlement.

3. The depth to a firm stratum or rock should be ascertained so that one may know definitely whether such material can be utilized practicably.

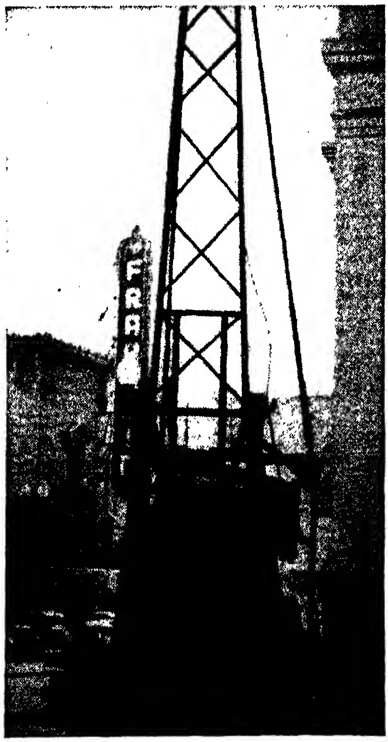


FIG. 3-2. A Raymond Concrete Pile Company's Gow-type wash-boring drill rig. *(Courtesy of Dames and Moore, San Francisco and Los Angeles, Calif.)*

4. Prove that dangerous material does not underlie a firm stratum.

5. Carry the exploration to such additional depth as the findings from the borings may indicate is essential in order to have proper information for planning the structure.

A wash boring is one means for making deep explorations. It is made by using such equipment as that pictured in Figs. 3-2 and 3-3. One method will be described:

A pipe casing is driven into the ground. A pump circulates wash water from a sludge-receiving tank through a hose, a swivel head, and a 1½-in.-diameter pipe inside the casing. This wash pipe has a fishtail bit used to loosen the soil, and the water washes the loosened material up and into the sludge tank, as pictured in Fig. 3-4. The bit is operated mechanically so as to produce vertical strokes 2 or 3 ft. long; in firm soils, the bit is rotated slightly at each stroke. These operations are continued until the penetration has proceeded as far as may be necessary or possible. The effluent can be sampled, as shown in Fig. 3-4, in order to determine the general

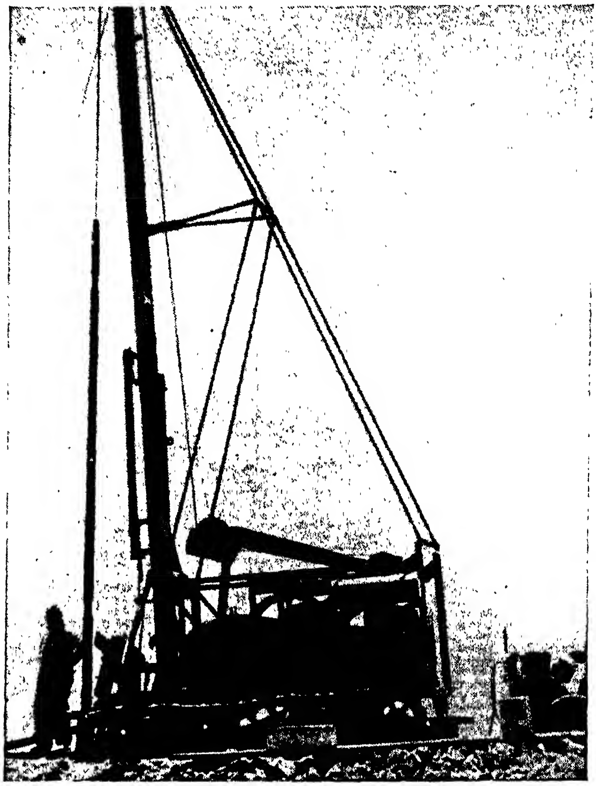


FIG. 3-3. A crawler-mounted cable tool rig. (Courtesy of Dames and Moore.)

constituents of the soil being penetrated; this, however, yields a completely disturbed sample. Nevertheless, experts at the business can secure data that are very helpful.

In Fig. 3-5 is shown the "log" of a boring that was made near New York City. The operators determined the limits of the various strata by watching and recording the material discharged in the effluent, and by noticing the differences in driving resistance. Depths are determined by computing the length of pipe that is below the surface. These points may then be located in elevation by taking levels to determine the elevation of the ground, or by interpolating the elevation of the top of the boring from a topographic map. The data in the fieldman's notes are then presented in this graphical form.

As it is usually very important to ascertain whether or not bedrock has been reached, it is desirable to drill approximately 10 ft. into the rock by means of a diamond or steel-shot drill. It is advisable to have the cores at least 1½ in. in diameter. In the case of Fig. 3-5, there was a small boulder above ledge rock. The recovery of only approximately 25 per cent of the

FIG. 3-4. A close-up of drilling procedure of a Gow-type wash-boring rig. (Courtesy of Dames and Moore.)

rock core indicated that the top of the bedrock was probably seamy and partially disintegrated.

"Undisturbed" samples of soil are obtained by removing the inner pipe from the wash boring, attaching a sampler or spoon somewhat like one of those in Figs. 3-6 and 3-7, then driving this into the unexcavated soil near the bottom of the casing. In Fig. 3-5 are shown the resistance to the penetration of the sampler, and the position from which each sample was taken; each sample was then sealed, numbered, and labeled with the depth to the place where it was taken. Figure 3-8 shows a sampler in use, whereas Fig. 3-9 pictures the preparation of soil samples for shipment to the laboratory. Samples 2½ in. in diameter are preferred because smaller ones

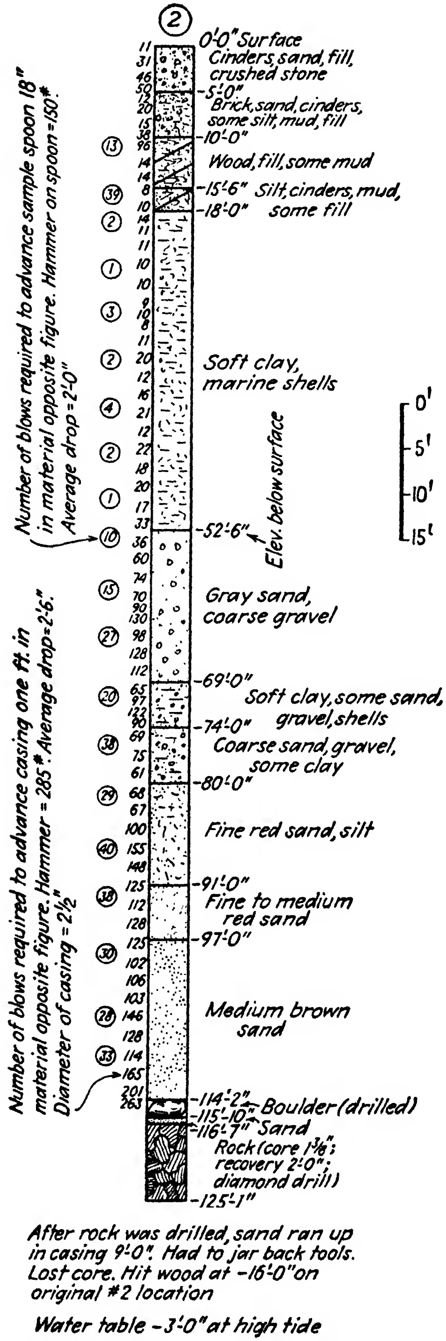


FIG. 3-5. Boring made for Anaconda Wire & Cable Co., Hastings-on-Hudson, N. Y. (Courtesy of Philip J. Henry, Inc., New York.)

are relatively more disturbed and may not be so suitable for standard testing equipment. Samples that are not taken skillfully may yield deceptive data. Of course, it is obvious that the driving of wash borings and the taking of soil samples in deep-lying coarse-gravel strata and boulders are difficult.

The elevation of the ground water is also an important item to be determined. This may be found by ascertaining the top of the water standing in the casing when it is left open for some time. However, in drilling through dense plastic clays, the relative impermeability of the soil may make it difficult for water to enter or leave the casing.

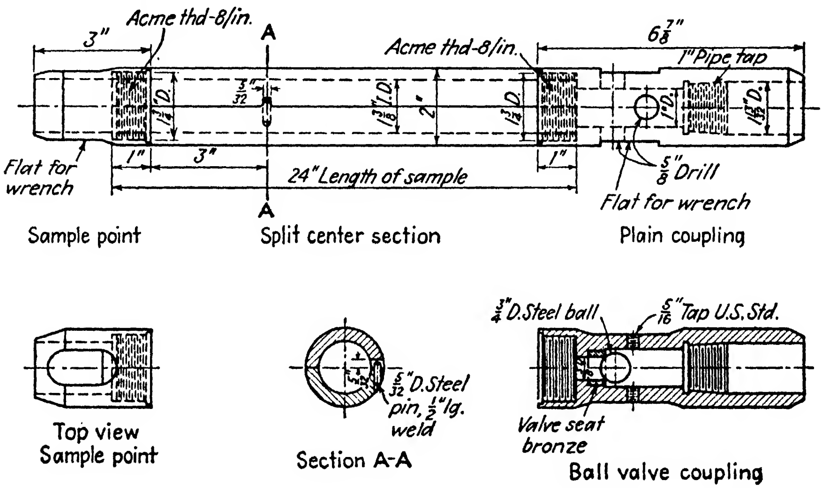


FIG. 3-6. One type of soil sampler. (Courtesy of the Raymond Concrete Pile Co.)

By a study of the materials encountered as shown in the log of the boring and by comparing the driving resistances, one may obtain a fair idea of the soil conditions under the ground at a particular spot. All or part of the undisturbed samples may be tested at a soil-mechanics laboratory to obtain a better idea of their qualities. If the foundation engineer does not perform the laboratory tests himself, he should confer with the one who does in order to obtain an expert opinion of the qualities of the soils and their probable suitability for the intended uses.

Samples of sandy and gravelly soil taken from the effluent from wash borings may indicate that the soil is finer than it really is. This may be caused by washing up the small particles whereas the coarser ones tend to remain in the hole and accumulate in the bottom. This situation should be watched. It can usually be detected when undisturbed samples are taken. Large borings 5 or 6 in. in diameter may be desirable when such soils are explored. Then some type of auger or excavator can be used to

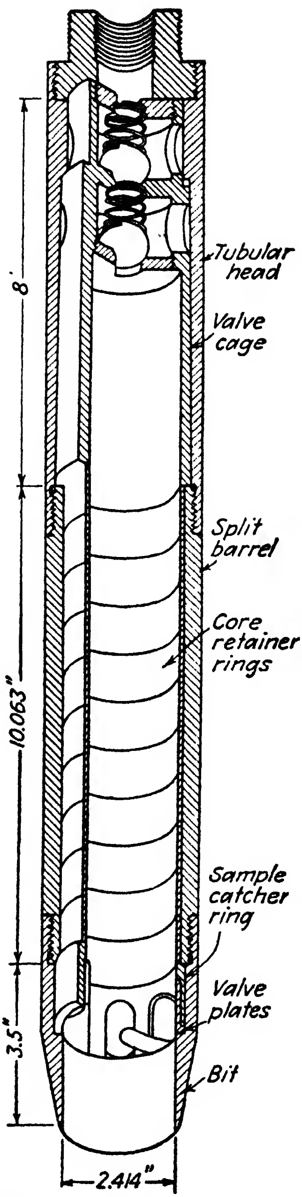


FIG. 3-7. A Dames and Moore underwater sampler.

obtain samples that show the proper grading even though they are greatly disturbed.

3-5. Soil profiles. From the logs of several borings in a row, one may draw soil profiles, similar to that shown in Fig. 3-10. These pictures are very instructive and should be drawn for various longitudinal and transverse vertical sections. Nevertheless, one should never forget how they were made and from what data. The borings were taken at specific locations only, as shown by the partial plan view and by the vertical lines on the profile. The borings therefore do not tell what lies between them. A picture is made by interpolating between the information given by the various borings, assuming the materials are in some reasonable and consistent arrangement. The intermediate conditions are obviously imaginary, but they give a helpful picture of the probable situation. Furthermore, the lines of demarcation between different strata are not likely to be so clear, and the method of their determination when the borings were made is by no means exact to the last inch.

In one case, borings for a vehicular structure were made approximately 150 ft. apart. Rock seemed to be at a fairly uniform elevation. However, when the excavation was made, it was discovered that two of the borings had been located on opposite edges of a ravine, which was approximately 30 ft. deep. It was filled with boulders and constituted the bed of an underground stream. As it was essential to place the foundations of the structure on the rock, many thousands of dollars were collected by the contractor for the extra work involved, probably much more than would have been needed if the presence of the ravine had been known in advance.

It seems that, once a draftsman has drawn a soil profile from the boring data, an engineer has checked it, and the drawing has been issued, the profile suddenly becomes accurate and authoritative in the opinion of the users

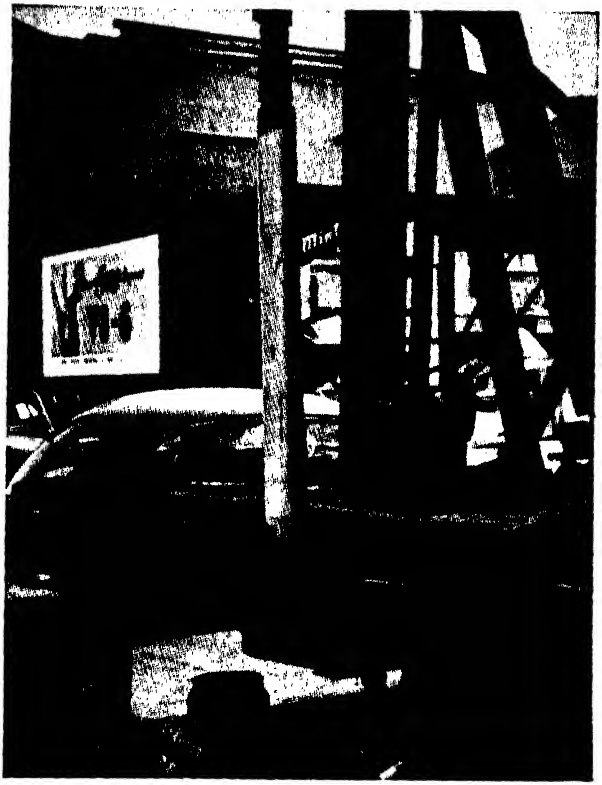


FIG. 3-8. A Dames and Moore sampler just pulled out of a test boring.

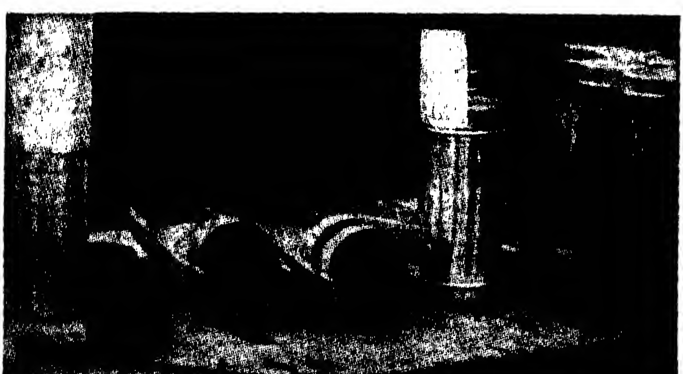


FIG. 3-9. A soil sample retained in the Dames and Moore sampler. The four rings of soil at the left are ready to be wrapped in waxed paper, to be labeled, and to be placed in a can for shipment to the soil-testing laboratory. The section of the sample in the foreground and the portion in the split barrel have been parted for visual inspection.

of the drawing. All persons concerned should remember that it is needlessly costly to bore every few feet in order to make sure that all the details about the subterranean conditions are known. This may be desirable in the case of a narrow bridge pier that must be supported upon rock, provided this rock seems to be exceedingly irregular or sloping. It is helpful if the soil profiles, as shown in this sketch, are made purposely with lines at the locations of the borings in order to show everyone which parts are based directly upon the logs of the borings and which are reasonable assumptions.

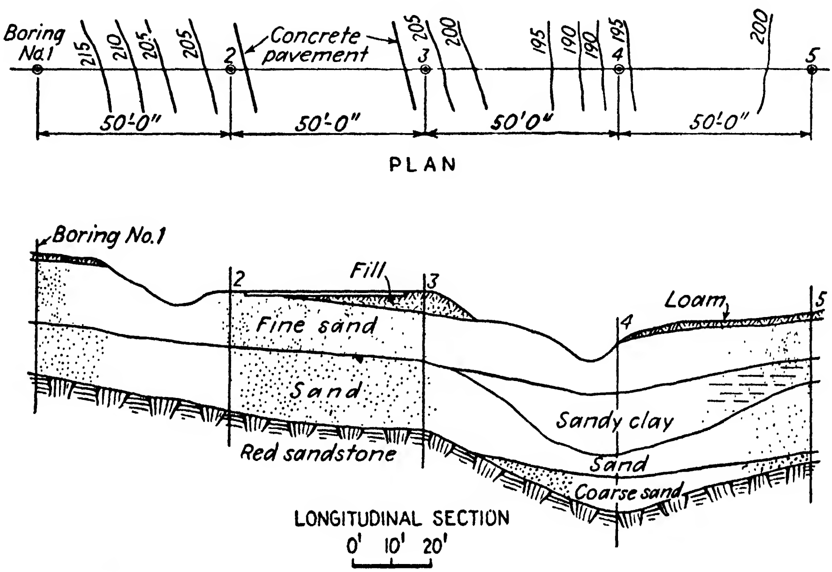


FIG. 3-10. Imaginary soil profile at site of proposed highway overpass. (Vertical scale may be exaggerated when necessary to clarify the picture.)

When soil profiles are used for information, the bidders on a contract should be told by a note on the drawing or by a clause in the contract papers that the information shown by the profiles is not guaranteed but is an interpretation of the data revealed by the borings. Furthermore, the soil samples and the logs of the borings should be available for all to see, so that each may have the opportunity to judge these matters for himself.

3-6. Load tests. Because so much depends upon the safety of a foundation, it is often advisable for the responsible engineer to make one or more load tests of the soil in the field before the final design of the substructure is made, provided the situation is such that these tests will furnish pertinent and important data. This may enable him to make considerable economies as well as to ensure safety. For example, if the available data without the results of direct load tests indicate that the soil has questionable supporting capacity, the engineer may believe that he must limit the allowable unit bearing pressure to 1.5 tons per ft.² whereas, if tests show that 3 tons per ft.² can be applied safely, he may be justified in using the latter in his design and thus save considerable in the size and cost of the footings. In other words, the lack of reasonably complete and reliable information may cause an engineer to be unduly conservative. He should not be willing to take unwarranted chances because, if he does so, he invites trouble for both the owner and himself. Any man may make an excusable mistake in deciding important questions; making a decision with the knowledge that the results are likely to be unsafe or unsatisfactory is not excusable in engineering. Furthermore, a moderate increase in the assumed allowable unit bearing pressure on the soil for design purposes will seldom reduce the cost of the substructure by any sum that is comparable to the seeming change because there is so much other necessary construction that has to be made approximately the same anyway; the footings are the only parts that are affected directly by this basic assumption as to the allowable bearing pressure.

Load tests of soil in the field may tell much about the shearing strength and compressibility of the ground that is tested directly. However, the test is insignificant compared to the forces that will be applied by the structure as far as major settlements and the behavior of underlying strata are concerned. For example, notice the following comments:

1. A load test on a surface stratum of clay or silt that is to support the weight of a structure directly is desirable in order to show whether local failure of such weak soils under the footings is probable.

2. A load test on a surface layer of sand 10 ft. thick will reveal the direct bearing capacity of the sand, whereas it will disclose little about the capacity of an underlying stratum of clay or silt because the sand spreads the pressure over such a large area of the plastic stratum that the unit pressure on the latter may be almost negligible.

3. The effect of a local load upon a small area of soil may not be extrapolated safely to apply to that of the same unit load placed upon a large area.

4. The effect of a short-time load on plastic soils should not be extrapolated to apply to that of the same unit load acting on a large area for a long time. This statement does not apply to firm granular soils.

When load tests of the ground at a given site are planned, it is best to know enough about the proposed structure, its location, and the elevation of its footings to enable one to test the soil at or near places where important loads will be applied, and at the elevation of the bottoms of future footings. This may require some excavation by hand. If the bearing plate of the testing equipment is rested directly upon the top of a relatively large area, as in Fig. 3-11(*a*), the test may be unfairly severe on the soil as far as a shearing and heaving failure is concerned because the ground has little lateral support. If the bearing plate is embedded somewhat, as in (*b*), the effect of the surrounding weight of soil may improve the test results and bring them closer to a comparison with actual conditions under the completed structure. On the other hand, if the size of the excavation is too small, as pictured in (*c*), and if the embedment of the bearing plate is too deep, the restraining effect of the surrounding soil may produce test results that appear to yield larger allowable bearing pressures than the conditions under the real structure justify. One should always bear in mind the small scale (or size) of the test compared to that of the proposed structure.

As a general rule, one may say that, the larger the bearing area of a load test on the soil, the more reliable are the results. In any case, the total

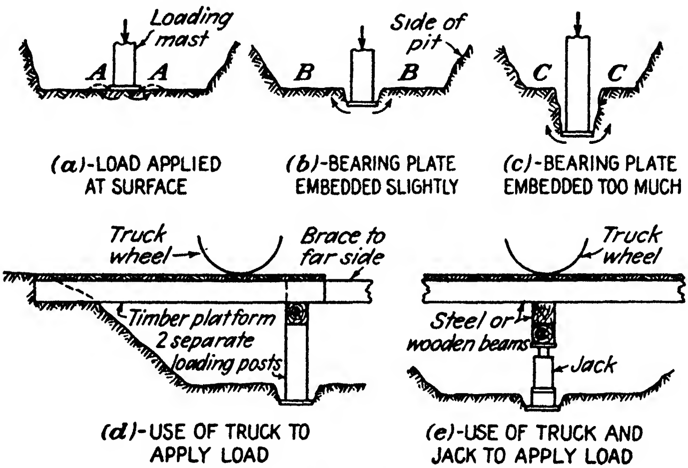


FIG. 3-11. Making load-settlement tests of soil.

applied load will probably be very small compared to that of the structure and, as stated previously, the test yields trustworthy data about the soil at and close to the bearing plate only. It is practicable to make a load test upon an area of 1 or 2 ft.², or somewhat larger. On the other hand, areas of 4 ft.² or more, when tested to the same unit pressure on the soil, generally require such a strong loading frame and such costly work in applying the required loads that they are not worth the extra cost.

Figure 16-7 illustrates the equipment used in making one such load test. This is described in so much detail in Art. 16-7 that similar information need not be given here. Besides the use of metal blocks, loads may be applied by means of the weight of water pumped into a tank on the loading frame, by filling a box with a known weight of sand, or even by running the weighed rear axle of a loaded truck onto beams that rest upon a loading frame with two supports, as in Fig. 3-11(*d*). This last method may be easy but it may not yield very accurate results. A more reliable method for quick loading may be that of running a very heavy truck or piece of equipment onto beams over a large test pit, then applying the test load by a hydraulic jack that reacts against these beams, as shown diagrammatically in Fig. 3-11(*c*). The tests should be made carefully and well, or they will not be worth their cost.

Load-settlement diagrams should be plotted for each test. One such diagram is pictured in Fig. 3-12. One should not be too hasty in testing

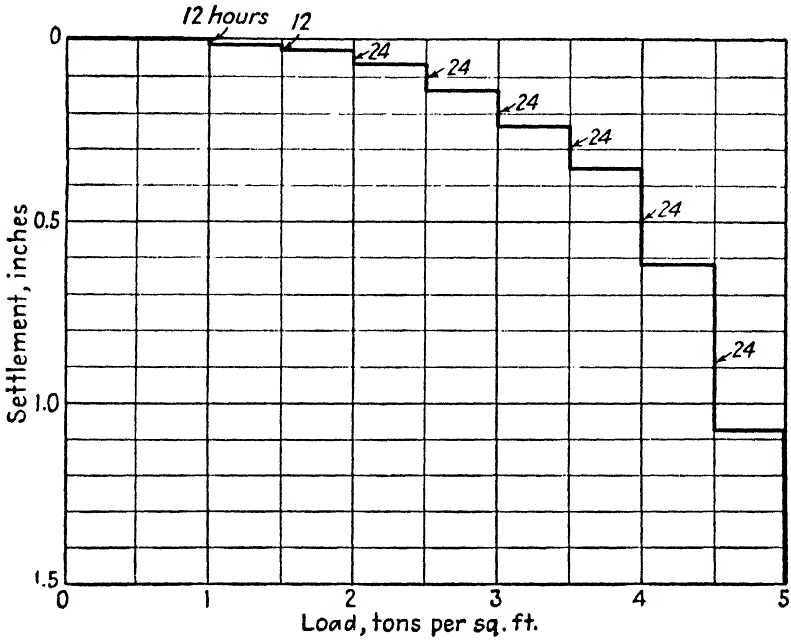


FIG. 3-12. Load-settlement diagram. (Note: Figures at vertical breaks give duration of time in hours between application of given load and that of the next increment of load. Ultimate bearing capacity appears to be 4.5 tons per ft.², but soil is so plastic that longer time should be allowed to obtain maximum settlement under smaller loads.)

plastic soils but should try to ascertain a definite yield point of the material so that the critical load may be determined and the proper safety factor of 1.5 to 2 secured when the maximum allowable unit bearing pressure is selected. If there is no well-defined yield point of the soil, the ultimate capacity of the ground may be taken as that unit pressure which causes or accompanies a total settlement of the bearing plate of some predetermined amount; for instance, 1 in.

3-7. Planning subsurface explorations. One may properly ask how much expenditure for subsurface explorations is justified in a given case. This cannot be answered specifically because it depends upon many things. Relatively large amounts may be warranted when failure or considerable settlement of the structure would be serious, when the lives of human beings depend upon the structure's safety, when dangerous soil conditions are discovered or seem to exist, and when the soil conditions present one of those borderline cases in which it is necessary to choose between a costly substructure of undoubted safety and a far cheaper but somewhat questionable type of construction.

In general, it is best for an engineer to have reasonably good information regarding the character, location, and weight of the contemplated structures to be built upon a specific site. He should then have a few widely scattered borings made for the purpose of ascertaining the character of the soil under various parts of the area that may be used. Of course, these borings should

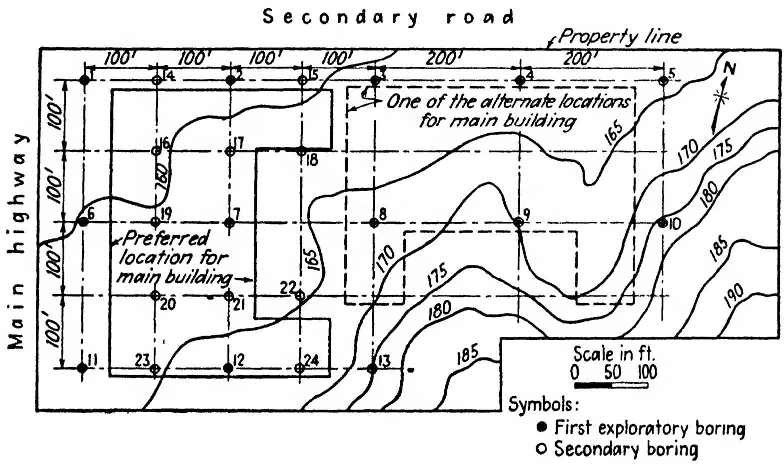


FIG. 3-13. Layout of borings at site of proposed industrial plant.

be closer together at the places where major structures are desired. A sample of such a plan is given in Fig. 3-13. This topographic map shows the outline and contours of a plot on which a corporation plans a manufacturing plant. The principal structure is to be a large heavy mill building; the other structures will be much smaller. The general layout of the equipment and main building has been made tentatively. The preferred location is shown by the solid outline; one of the alternate locations, by the dotted lines. Very little is known of the details of the soils beneath the surface. Therefore, a series of borings 200 ft. apart each way is planned as the first step in the exploration; these are numbered 1 to 13, inclusive.

If the preferred location seems to be the best and if the soils are excellent and uniform, no further borings need be made. If the conditions at the preferred location are not uniform or if the findings show that more borings are desirable, secondary borings, 14 to 24, may be made. On the other hand, if borings 1, 2, 3, 6, 7, and 11 reveal the presence of weak soils, whereas those farther east show that the ground is much better there, then the location shown dotted may be investigated further.

Most of the preliminary borings may extend to a depth of 50 ft. below the surface unless hard material is encountered at higher elevations. At least one or two borings, such as 7 and 9 of Fig. 3-13, should go down 100 to 125 ft., especially at the preferred locations of heavy structures, if no strong bearing is found at a higher elevation, in order to determine whether firm materials exist within the reach of piling.

It is generally advisable for an engineer to be on the job during the making of the borings, or to have some other engineer observe the operations for him. It is also desirable to have the contractor prepare daily progress reports so that the engineer in charge of the planning may have the information as quickly as possible and take action if necessary.

The boring contract should be made with provision for a unit price per foot for extra borings that may be desired, in order to enable one to explore more thoroughly under any areas that are found to have soils of questionable value, any areas that appear to offer promise of better materials, and other locations where further information is desired. A unit price should be provided also for taking extra undisturbed soil samples that may seem to be needed as any of the borings are made. This enables the engineer to obtain more borings and samples if essential, doing so at known costs.

For example, at the site for one industrial plant, two of the preliminary borings revealed what seemed to be a buried stream bed filled with silt and clay, whereas the others showed good sand; therefore, a series of shallow borings was made in order to determine the course of this localized weak material across the site. The main structure was then relocated slightly in order to minimize the number of foundations that had to be extended down through this soft material. At the location of a group of deep pits that were to be a part of the plant, one boring was carried down below their bases in order to learn whether rock would be encountered, because the excavation of rock would add greatly to the cost of construction.

At another site, 30 ft. of sand was underlain by at least 20 ft. of very fine sand—a rock flour. One of the structures was to be a heavy blast furnace. Therefore, at the preferred location for the furnace, another, deeper boring was made, and rock was located at a depth of 83 ft. This showed that piles could be used if it was decided that the rock flour was untrustworthy.

When preliminary borings are made for a bridge whose location is not fixed by other considerations, it may be desirable to make a series of borings along various possible alignments in order to determine which one is best from the standpoint of foundations. The conditions at the locations selected for the piers and abutments along the best alignment can then be explored more carefully, perhaps by means of a group of borings spaced 20 or 30 ft. on centers both ways. If the customary 10-ft. core borings at the location of a heavy pier show that the rock is badly seamed or poor in quality, some of these borings may be carried to considerably greater depths in order to locate sound bedrock.

3-8. Importance of subsurface conditions to the planning of structures. It is important for an engineer to know the details of subsurface conditions at a site before he completes the design of the structure. There are many reasons for this. These conditions may also have great effect upon the planning of construction procedures. Unanticipated conditions generally cause additional trouble and expense.

Nonuniform plastic soils may limit the use of continuous and rigid-frame structures because of the danger of serious unequal settlement, thereby

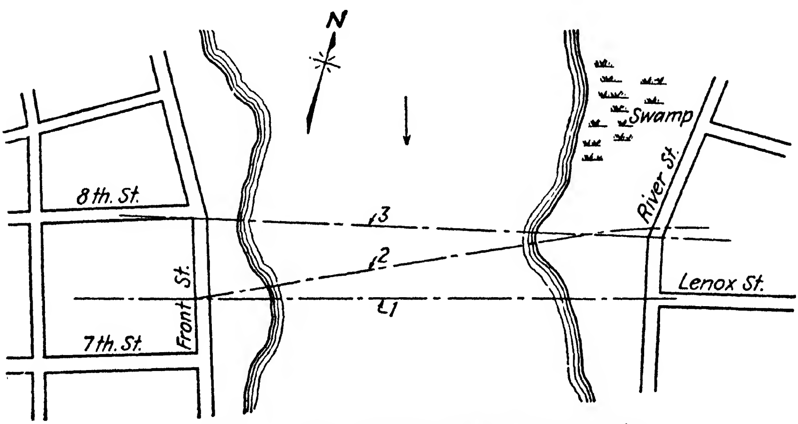


FIG. 3-14. Illustration of alternate alignments for a bridge.

causing the designer to plan for the use of simply supported structures, or those utilizing some other determinate type such as cantilever bridge construction. These soils may prevent the use of arches on account of the likelihood of horizontal yielding of the earth. Heavy multistory structures may be inadvisable because of probable settlement and cracking. Lightweight flexible structures of steel or wood may be preferable so that distortions may occur without serious harm.

In some cases, the shape of a building may have to suit the area of good soil upon which it can be placed. The span of a bridge, or several spans in a long structure, may have to conform with the conditions under the ground. For example, rock may be at or near the surface in one or more places so that these are the most advantageous locations for piers or abutments; silt may have filled certain old channels that should now be avoided if possible; and boulders may cover the rock at certain places so that the sinking of caissons and the driving of piles would be hazardous there. Weak soils may require the use of many small light columns and spread footings on the one hand or, on the other, the use of long spans with a minimum number of foundations but with each one extended down to firm material by means of piles or caissons. Great engineering skill may be required to secure the best coordination of architectural features, utility, safety, construction, and economy. This is engineering planning.

It often happens that some one wishes to build a structure of a certain character, of a specific size, and of a particular material on a given site. Unfortunately, it sometimes happens that the plans have progressed considerably before the subsurface conditions at the site are explored. The engineer is then faced with the problems of planning a foundation that will hold the structure safely with a minimum reasonable cost, of persuading the owner to modify his plans to accommodate the existing conditions, or of convincing the owner that he should locate the structure upon some other more favorable site. If the engineer accedes to the owner's wishes against his own best judgment and if the structure is not satisfactory, the engineer will still be blamed for the results because, in time, the owner is likely to say, "Why didn't you explain these things to me more fully and show me that the situation was so bad?"

Many times proper subsurface explorations have yielded information which affected the design of a structure and which enabled the engineer to save far more than their cost by avoiding design revisions and charges for extra work not included in the original contract. Many times the reverse has happened when adequate data were not secured.

Consider these cases in addition to those cited previously:

1. The primary crushing plant at a mine was to be a heavy concrete structure about 80 ft. high from the base to the main working floor, and a steel superstructure was to be above that. The concrete part was to be in a niche excavated in a rocky mountainside. Diamond core borings, made to the level of the invert of the substructure, revealed that much of the upper part of the rock was seamy with disintegrated claylike material in the seams. Furthermore, these seams dipped downward at about 70° from the horizontal, creating the possibility that the uphill side of the cut would slide into the excavation. A truck roadway to be at this uphill side of the structure made it inadvisable to remove these upper layers behind the structure completely. Under these conditions, the structure was made as narrow as possible so that the niche to be blasted out of the rock would probably be small enough to enable the remaining rock to arch around the cut. The structure was then made longer in a direction perpendicular to the hillside, and it was designed with two outer walls and two inner partitions of heavy reinforced-concrete construction which bore against the rock and acted as huge buttresses that would not let the rock start to move. These considerations were the starting points in the planning of the structure which is shown under construction in Fig. 3-15.

2. Another heavy industrial structure of a character somewhat similar to that just described was to be located in "solid rock." Adjacent hillside cuts and other excavations for related structures were made, then, belatedly, a core boring was driven down to the bottom of the proposed structure. Here was found an old inactive fault zone of shattered rock cutting on a slope through the region at and just above the proposed invert. Neighboring work had proceeded so far that it was not practicable to choose another location. Although resting a heavy structure across a fault is not

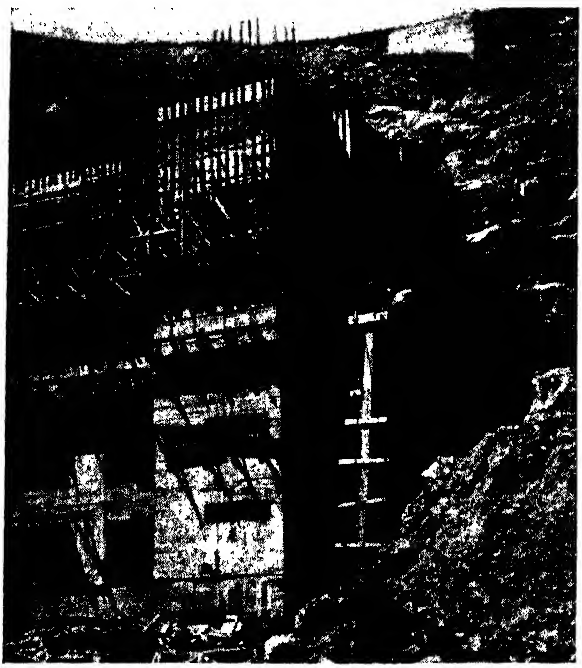


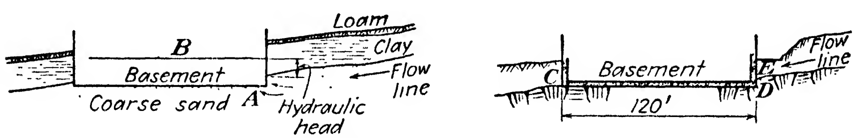
FIG. 3-15. A primary crushing plant under construction on a mountainside. The structure itself is a heavy reinforced-concrete buttress designed to prevent any slippage of the rock behind it. (Courtesy of Cananea Consolidated Copper Co., S.A.)

recommended practice, it was done in this case because a major earthquake seemed to be improbable and, if one should cause a serious movement of this fault, the rest of the plant would probably be ruined also. However, since the structure was very heavy, the design was made so as to provide a thick mat of reinforced concrete at the bottom to spread the loads. Since the fault would probably be a source of water under high pressure, means were taken to provide drainage and to prevent the building up of hydrostatic pressure. These things were provided easily because known in advance.

3. A heavy building was designed to be supported upon rock. The "top" of the rock was located by means of wash borings, but core borings

were not made. It was discovered during construction that the boring at one corner had hit a boulder. The foundations in this vicinity had to be revised and carried down much deeper than intended, with a large and just claim for extras by the contractor. If the contractor and the engineer had known the conditions beforehand, both the design and the method of construction could have been adapted to them with much saving in the total expenditure.

4. Sometimes an investigation of ground-water conditions will avoid much trouble. Figure 3-16(*a*) pictures one kind of possible situation. Here the substructure was to be supported upon sand. When the clay stratum was penetrated, it was discovered that the ground water in the sand layer had been trapped under this clay, and the water was under sufficient pressure to cause it to enter near *A* and to fill the excavation to *B*.



(*a*)- EXCAVATION INTO A POROUS, WATER-BEARING STRATUM (*b*)-FOUNDATION ACTING AS A DAM

FIG. 3–16. Examples of troubles caused by ground water.

Wellpoints had to be driven to the sand around the excavation, and heavy pumping was required to remove the water fast enough because of the large volume of the supply. The entire basement had to be redesigned to resist the pressures and attain waterproofness.

Even in a case like that shown in Fig. 3–16(*b*), ground water may cause trouble. Here the basement of a building several hundred feet long was built on rock. The soil explorations were made during dry weather when the ground water was negligible, and no attempt was made to ascertain conditions at other times. The foundation wall above *D* served as a dam to impound the water flowing down to *E*. This water would eventually flow around the building, but it caused dampness and some leakage that were objectionable and costly to rectify. A knowledge of the conditions would have enabled the designer to plan for the waterproofing of the uphill wall, for the installation of collecting drains along the side at or above *D* and around the ends of the basement, or for the building of a few drains from *E* under the basement floor to *C*.

In one situation explored during the dry season, a test pit was dug 15 ft. deep so that the soil could be examined carefully. This revealed occasional thin layers of small gravel and coarse sand that were devoid of fine particles. The engineers concluded that these strata were clean because they were small watercourses during wet weather. This proved to be the case.

5. A harbor was to be improved during the recent war. A cursory examination of the general vicinity indicated a muddy bottom and shallow water. Because of the great rush, soundings were made to determine the depth of the water at the site of the pier; however, no time was spent with borings. A big hydraulic dredge was dispatched to the site, and material was assembled for the construction of a pier to be founded upon the assumed materials. When the dredge arrived and excavation work was started, it was discovered that solid rock lay under a very thin stratum of mud throughout the bay. Of course, the entire plan had to be changed radically, and much valuable time was lost.

3-9. Value of investment in engineering. The emphasis upon careful planning and study of foundation problems serves to bring to the reader's attention the question of justified expenditures for engineering in general as well as for the exploration of a site. Sometimes one hears that the engineering costs of a certain big job were 4.45 per cent of the cost of the completed work, whereas the engineering cost on some other project was 7.82 per cent of the cost of construction. This gives the impression that the engineers connected with the first project were much better and more capable than those who worked on the second job. This may be utterly fallacious for many reasons, some of which are the following:

1. Engineering costs vary greatly because of the vastly different character of the work to be done.

2. Engineering costs are reduced in terms of the percentage of the construction cost when there is considerable duplication of parts. For example, a steel-frame mill building 100 ft. long with 5 typical 20-ft. bays on uniformly good soil is likely to cost almost as much for the exploration, the design work, and the drafting as would a similar building 400 ft. long with 20 of the same typical bays. On the other hand, the construction costs of these two buildings would vary somewhat as their lengths.

3. In general, small jobs involve more engineering expense, relatively, than do large ones of the same basic character.

4. Engineering work that is done hastily and carelessly may be cheap in itself but expensive to the owner. For instance, a complete project may have cost $500,000, whereas the engineering was kept to a minimum, say $25,000, or 5 per cent of the total cost. On the other hand, the engineering might have been done with careful study and great skill. with an expenditure for engineering of $40,000 but with a *total* cost for the same project of $460,000, giving a relative cost of engineering of approximately 8.7 per cent. Nevertheless, the owner in the second case would have saved $40,000 in construction costs, and his investment of $15.000 more in engineering would have paid excellent dividends in cash as well as in satisfaction.

5. Sometimes various engineering expenses are hidden in the construction cost. One should be sure that he understands the bookkeeping involved in the cases that he compares.

If a project is satisfactory because it has been well planned and carefully worked out, how is one to compute the monetary value of the engineering services compared to the "cheap" job with which no one is pleased? Seldom is an owner grateful for small savings—and they are relatively small—made at the sacrifice of safety, utility, and the general satisfactoriness of the structure. The importance of careful investigation and study

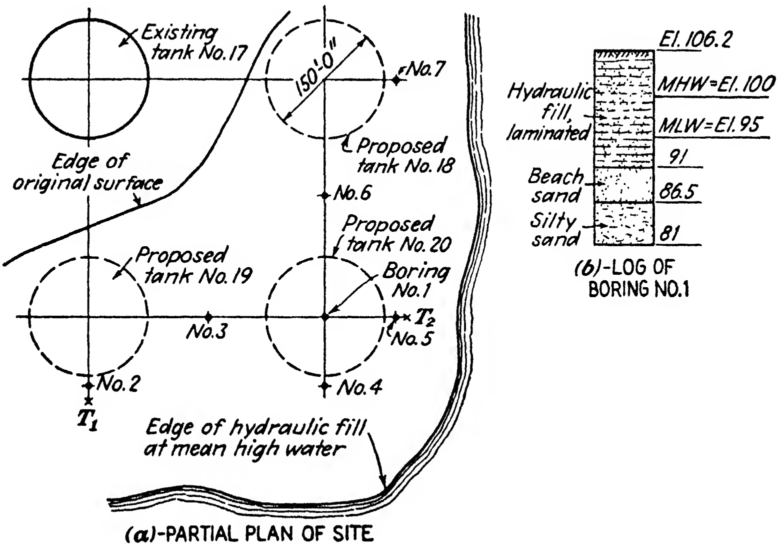


FIG. 3-17. Data for exploration of site for some new oil tanks.

of foundation conditions and of all other engineering problems connected with a job is self-evident.

3-10. Example. An oil company wishes to build an addition to its tank farm alongside tidewater. Recent dredging operations have produced an extensive hydraulic fill next to the company's property, and the area of this new land is adequate for three tanks and their dikes. The surface of the ground is approximately 6 ft. above mean high water (M.H.W.). A typical proposed tank is 150 ft. in diameter and 40 ft. high. A partial plan of the site and the log of exploratory boring No. 1 are shown in Fig. 3-17. Is it safe to support these steel tanks primarily upon a sand cushion placed directly upon the fill?

First, one should obtain a general idea of the magnitude of the things with which he is dealing. Some data for this case are the following:

1. A 40-ft. depth of oil will cause a unit pressure of nearly 2,500 p.s.f. on the soil.

2. A 150- by 40-ft. tank, when filled, will cause a total load of nearly 22,000 tons.

3. At 10 cents per pound for steel and 10 cents per gallon for oil, such a full tank would be worth over $600,000.

When one is faced with the problem of founding a heavy important structure upon hydraulic fill, he may have something to worry about. He should remember that such fills are likely to contain lenses of material of different characteristics and to be poorly consolidated. The coarse materials generally settle near the point of discharge, whereas the finer particles spread out widely. From time to time, the discharge pipe is usually moved to different locations. This may cause local deposits of coarse materials to be above or below strata of fine loosely compacted soil. The magnitudes of the differential settlements are difficult to predict. If muck, silt, or soft clay underlies the hydraulic fill, this will add to the uncertainties. If the water-table is near the top of the fill, heavy pressure over a large area may cause plastic materials and very fine sand to flow out from under the loads.

Here the boring shows that there is some silt in the sand stratum below the hydraulic fill. The driving record indicated that this layer permitted penetration even more easily than did the fill. However, what are the conditions and probabilities elsewhere?

The value of three tanks and the cost of deep foundations to guarantee safety justify the expenditure necessary to explore the soil thoroughly. The one boring indicates that the sand 15 ft. below the surface is probably adequate if the upper strata can transmit the load to it. Since the danger seems to lie in the upper 15 ft., it appears that at least six more shallow (25-ft.) borings should be made at locations 2 to 7, shown in the figure, in order to obtain a better idea of the probable value and character of the soils. Since none of the original borings was deeper than 25 ft., one new one, like No. 4, might well be carried down 25 to 50 ft. farther in order to make sure that no dangerous plastic materials underlie the sand. This is advisable because the extent of the loaded area under a tank is so large that there is a relatively small rate of decrease in unit pressure caused by the distribution or spreading of pressure by the soil.

Although load tests of the upper strata cannot simulate the effect of such extensive loads as those of the proposed structure, a bearing test of both the fill and the underlying stratum should be made near one or two of the borings, as shown in Fig. 3-17 by the symbols T_1 and T_2. Further tests should be made if the borings and the results of the first load tests indicate that they are necessary.

PROBLEMS

3-1. Draw a soil profile along the center line of the borings shown in Fig. 3-18. Use a vertical and horizontal scale of 1 in. = 30 ft.

3-2. Draw the soil profiles along the lines connecting borings 1, 2, and 3, also 4, 5, and 6, in Fig. 10-53 (Chap. 10). This is alongside a river so that scour from floods through the ages has washed away portions of the ground, then other deposits have been made. It illustrates some of the troubles encountered in interpreting boring data.

3-3. An electric power company is planning an addition to a generating station at the water front of an important city. The extension is shown by the hatched

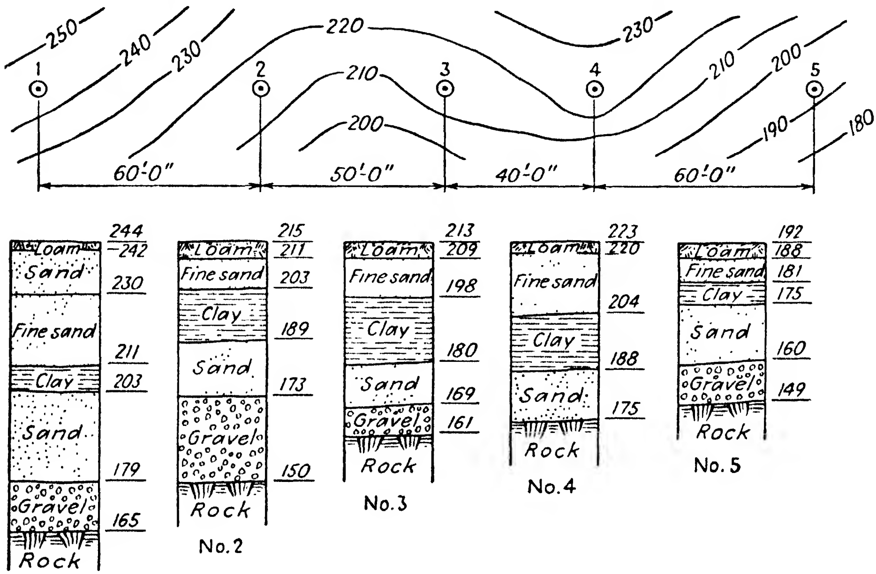


FIG. 3-18. Logs of borings.

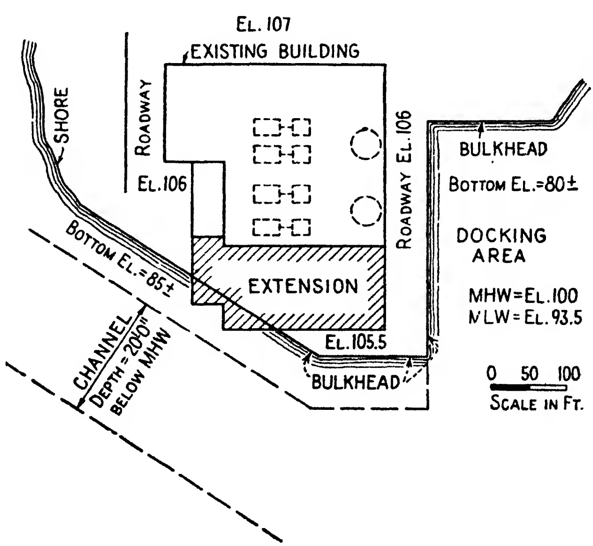


FIG. 3-19. Plan at a power plant.

areas in Fig. 3-19. Determine the locations, type, and depth of borings to be made before the foundations are planned. The present building is supported upon piles that penetrate through a few feet of fill and 25 ft. of harbor silt to fine sand.

3-4. A friend is planning to buy a lot on which to build a one-family brick house. He has asked you to advise him in the matter. The lot is along a suburban road and

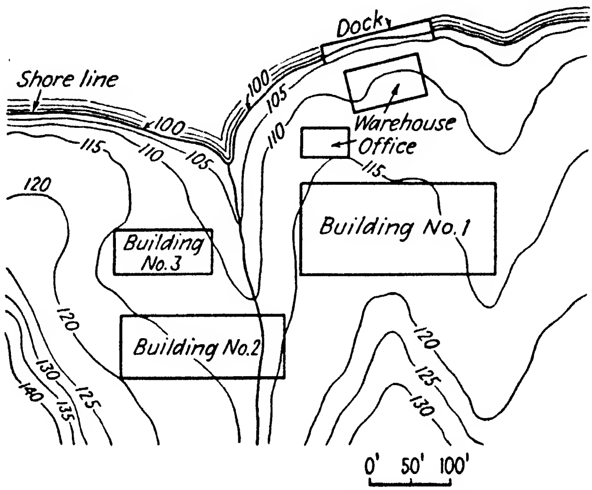


FIG. 3-20. Plan of a proposed industrial plant.

has a width of 100 ft. and a depth of 200 ft. Near one front corner is what appears to be the top of a flat boulder. Around it for several feet are a few scraggly bushes and weeds. The ground slopes downward diagonally from this corner to the opposite rear corner, which is approximately 6 ft. lower. Crossing this corner there is a small brook amidst alders and a few swamp maple trees. Miscellaneous trees and bushes are scattered between these two corners. What will you do, or tell your friend to do, before purchasing the property?

3-5. Refer to Fig. 3-20. Plan and specify the borings to be made so that preliminary studies can be made to determine the general types of foundations for the structures.

4

SOME PRINCIPLES OF FOUNDATION ACTION

4-1. Introduction. Various assumptions and general principles regarding the action of soils under loads are described in this chapter. These should be understood thoroughly so that they can be used in the planning and the detailed designing of foundations for a great variety of structures built in many different situations. Undoubtedly there are differences of opinion held by various individuals regarding these assumptions and the proposed technique of procedure. Yet, if rightly used as tools, the principles set forth will yield reasonably satisfactory results, will enable the reader to visualize his foundation problems more clearly, and will help him solve these problems safely.

Probably it cannot be stated too often that the *exact* properties of soils are unknown, reasonable approximations of them are generally the best that are available, and these properties vary greatly. A given stratum is likely to be a mixture of materials that give to it some of the qualities of each of the major constituents, the properties of each being modified by those of the other different materials. The degree of compaction and the moisture content affect these properties, too. Therefore, the factor J—meaning judgment—is always to be inserted in the equations used by the foundation engineer.

4-2. Distribution of pressure. Assume that a footing like that shown in Fig. 4-1(a) rests upon the ground and supports a total vertical load P. The footing is in equilibrium; therefore, the resisting pressure under it must equal $-P$. Is this pressure distributed uniformly over the area of the bottom of the footing, as pictured in Sketch (a)? Is the intensity higher near the edges, as in (b)? Or is it larger in the center, as in (c)? A relatively compressible soil *may* produce pressures somewhat like those in (b); a rather incompressible soil, like those in (c). Probably none of the results is *exactly* as pictured in (a). Nevertheless, for practical purposes, this uniform distribution may be assumed for the design of the footing itself and for an analysis of the pressures in the soil under it, because this assumption is probably as accurate as are those that must be made concerning tl action of the reinforced-concrete footing, the soils, and the magnitudes (the loads.

If the footing is on plastic soil, the inequalities of pressure may equali: gradually because of the greater compaction of the areas that are loade more heavily. If the footing is on granular soils that can withstand tl

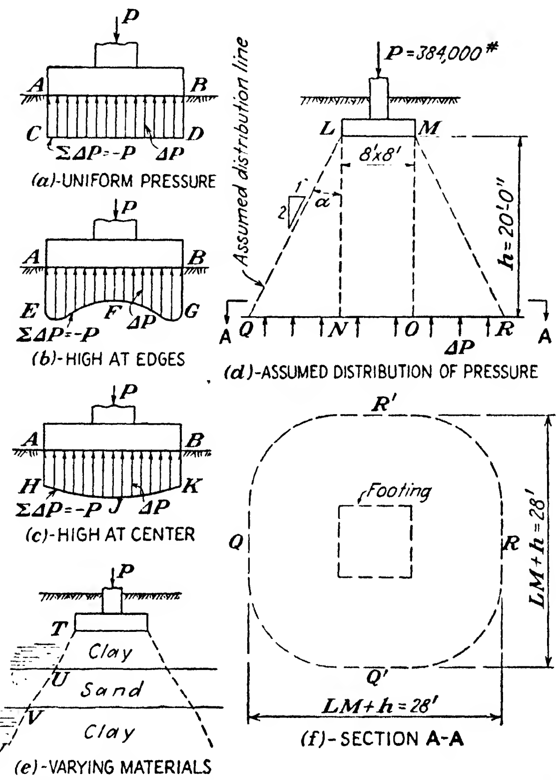


FIG. 4-1. Assumed distribution of pressures into soil under a footing.

necessary pressure, an increase of intensity near the center of the footii above the average pressure, will not harm the footing; if the soil yie slightly under the center, the edge pressures tend to rise toward the avera,

When the footing in Fig. 4-1(d) is loaded, it is obvious that it will ı merely compress a rectangular prism of earth pictured by $LMON$. She ing stresses on the soil within LN and MO will affect the soil outside of th planes and cause a spreading of the load. The angle α will depend uŗ the shearing strength and cohesion of the soil: furthermore, the surface (may not be clearly delineated at all. In the case of weak soils with sm internal frictional resistance and low cohesive strength, α will be smal than for firm granular materials. As a general average, it is sufficient to assume that distribution of the load will spread out at a slope of 2:1, as shown in Sketch (*d*). When a series of differing strata is present, the distribution may vary as pictured by the broken line *TUVW* in Sketch (*e*); however, this refinement is unnecessary and, undoubtedly, still inaccurate.

The area *QQ'RR'* in Fig. 4-1(*f*) indicates that, as the depth h increases, the assumed affected area of soil approaches a circle with a diameter of $LM + h$, and that the intensity of pressure ΔP decreases rapidly as h increases. For purposes of study and comparison of designs, it is sufficiently accurate to assume that the resisting area retains the general shape of the footing. The corners may be considered as quarter circles if one prefers to do so. The error caused by this assumption is greater in the case of small footings than for large loaded areas. However, when the depth of the plane being investigated is great with respect to the dimensions of the footing, the intensity of pressure is probably decreased to unimportance anyway. The pressures that exist outside this assumed pyramid are probably relatively small.

For example, if P in Fig. 4-1(*d*) is 384,000 lb., the footing is 8 ft. square, and h is 20 ft., the intensity of pressure p at LM is 6,000 p.s.f., whereas at QR it is approximately $384{,}000/28^2 = 490$ p.s.f. At such depths, it is probable that the unit pressure at the center of the assumed affected area of the plane QR is greater than the average, perhaps by as much as 50 per cent. Nevertheless, this is not serious in the case of small foundations for which the critical soil pressures are directly under and close to the footing. Large heavy structures and weak substrata are discussed more fully elsewhere. The principle, however, is illustrated by Fig. 4-2 where *ACEDB* pictures a

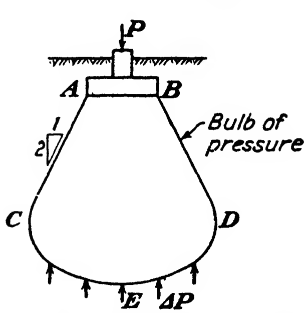


FIG. 4-2. Bulb of pressure assumed for purpose of visualization.

bulb of pressure bounded by a curved surface that represents an assumed locus of points of equal pressure. This method of picturization is useful in studying the action of soils.

Now compare the effects of a large loaded area and a small one upon the unit pressures assumed to exist at a plane in a stratum below the structures.

Figure 4-3(*a*) pictures a footing *AB* and large structure *GH*. Assume that the intensity of pressure at *AB* and *GH* equals 4,000 p.s.f. According to the assumption of 2:1 distribution, the intensities of pressures at 20- and 40-ft. depths below these structures are the following:

$$\text{At } CD,\ p_{20} = \frac{400{,}000}{30 \times 30} = 445 \text{ p.s.f.}$$

$$\text{At } JK,\ p_{20} = \frac{40{,}000{,}000}{120 \times 120} = 2{,}780 \text{ p.s.f.}$$

$$\text{At } EF,\ p_{40} = \frac{400{,}000}{50 \times 50} = 160 \text{ p.s.f.}$$

$$\text{At } LM,\ p_{40} = \frac{40{,}000{,}000}{140 \times 140} = 2{,}040 \text{ p.s.f.}$$

These estimates indicate that *AB* does not have any important effect upon these two lower strata, whereas the pressures from *GH* are reduced only about 50 per cent at a depth of 40 ft. Therefore, a weak stratum underlying a large heavy structure may constitute a real hazard.

In the case of a row of closely spaced footings like that shown in Fig. 4-3(*b*), the assumed "cones or pyramids of distribution" overlap. If the pressure at the footing *SU* is 10,000 p.s.f., that over *WT* alone would be $640{,}000/28^2 = 820$ p.s.f. Because of the overlapping at *VT*, the pressure in this area would seem to be doubled, 1,640 p.s.f. However, it is not

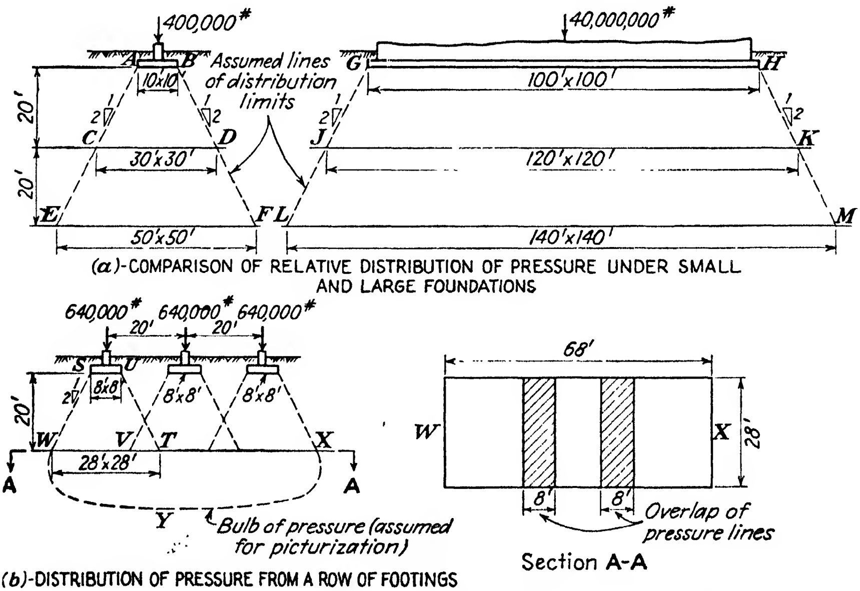


FIG. 4-3. A study of the distribution of pressures in soils under various foundations, assuming 2:1 distribution.

reasonable to suppose that the unit pressure from W to V is one value whereas that from V to T is twice as much. In such a case, it is best to assume that, when small overlapping occurs, the average unit pressure at WX is $\frac{3 \times 640{,}000}{68 \times 28} = 1{,}000$ p.s.f., but that the intensity through the central portion will be larger and that at the ends and edges will be less, somewhat as shown by the bulb-of-pressure line WYX. When there are large overlaps, it is more conservative to add the two computed pressures.

The Boussinesq equation

$$\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1 + (r/z)^2}\right]^{5/2} \tag{4-1}$$

may be used to estimate the vertical pressure at some point at depth z and horizontal offset r caused by a concentrated load Q. This formula assumes that the earth mass is homogeneous, elastic, and isotropic, which it seldom ever is. On the other hand, it gives a reasonable idea as to the probable magnitude of the vertical pressure, and this information is very useful.

On the other hand, what is usually wanted is the total unit pressure at one or more points caused by a number of local loads or by distributed loads acting over one or more significant areas. The Newmark circle[1] shown in Fig. 4-3A is a way of approximating the theoretical pressure at a point caused by uniformly distributed loads over a large area without having to use the Boussinesq formula directly, although the diagram is based upon this formula. The distance z is the vertical depth of the point to be investigated below the bottom of the bearing area. This distance is drawn to some scale ab, and this is then used as a unit distance. With a convenient point as center, circles may be drawn with radii equal to the corresponding proportions of z that are shown in the table in the illustration. Twenty sectors are then drawn with central angles of 18°. A diagram of the loaded area is also drawn to the scale of z on a separate piece of thin tracing paper. This latter sketch is to be laid over the circles with their centers at the position of the selected point z in the plan. Each area between two adjacent radial lines and circumferences is called an *influence unit*. The number N of these units included in whole or in substantial part within the outline of the loaded area is determined. If the uniform load on the bearing area is q p.s.f., then the vertical pressure at z is computed as

$$\sigma_z = q \times N \times 0.005 \qquad \text{p.s.f.} \tag{4-2}$$

To illustrate the procedure, assume that an L-shaped apartment house is planned. At a depth of 40 ft. below the basement slab is the top of a 10-ft. stratum of soft reddish clay that may cause harmful settlement.

[1] Terzaghi and Peck, "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York, 1948.

The average bearing pressure directly under the basement slab is 1,500 p.s.f. The intensity of vertical pressure at the center of this layer caused by the structure is to be estimated. The pressure under the vertex of the reentrant angle of the "inner" walls will be computed as one specific case.

The distance *ab* of Fig. 4-3*A* therefore represents the 45 ft. to some scale. The outline of the building drawn to this scale is shown by the dotted lines

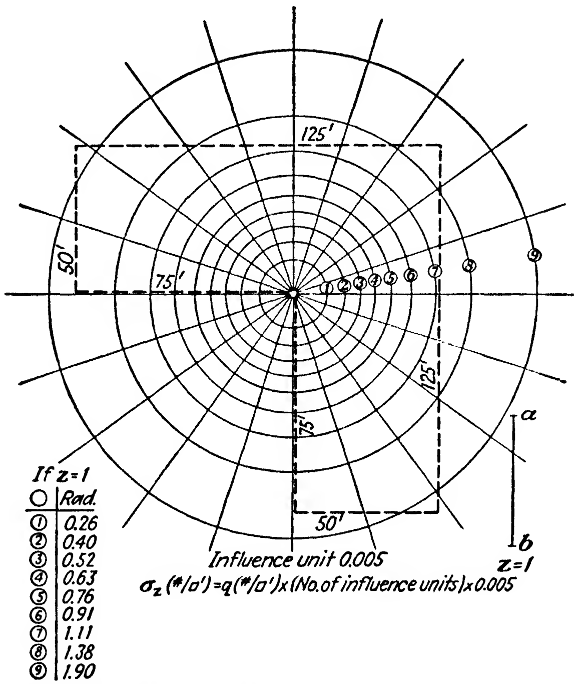


FIG. 4-3A. Newmark influence chart for vertical pressure at a point. Draw plan of loaded areas to a scale such that Z equals ab. Place point of σ_z on plan at center of Newmark circles. Count influence areas. (Courtesy of Nathan M. Newmark.)

in Fig. 4-3*A*, and it is placed with the chosen corner at the center of the circles. The approximate total of the influence units is taken as $N = 123$. Therefore,

$$\sigma_z = 1{,}500 \times 123 \times 0.005 = 920 \text{ p.s.f.}$$

Computation of the pressure at this point by the 2:1 method with overlap of assumed distribution areas gives

$$\frac{1{,}500(50 \times 125)}{(50 + 45)(125 + 45)} + \frac{1{,}500(50 \times 75)}{(50 + 45)(75 + 45)} = 1{,}070 \text{ p.s.f.}$$

This comparison of the results of the two computations is made for interest only. It is not to be considered a check of the 2:1 method because problems may be compared the results of which vary widely.

4-3. Shearing of soil. When a soil is subjected to the load of a structure, as in Fig. 4-4(*a*), there is the tendency for it to squeeze out from under the load. It may try to go one way as shown in (*a*), or, theoretically for a symmetrical structure and perfectly homogeneous soil, it may move out both ways as in (*b*). If real movement of the ground does occur, there will be a sliding movement along some such surfaces as *ABC* and *DEF*, so that the shearing and cohesive resistance of the soil must be overcome, the structure will settle, and the soil will be heaved up as indicated by the dotted lines. This action is especially to be guarded against when the unit pressures under the structure are large. It is obvious that sand and gravel, because of their large internal frictional resistance, are not so likely to fail in this matter as are plastic soils.

A structure placed directly on top of the ground may squeeze out plastic soils rather easily. If the bottom of the structure is depressed, or

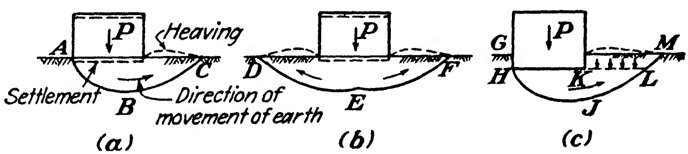


FIG. 4-4. Principle of failure of soil in shear when under pressure.

embedded, some appreciable depth *GH*, Fig. 4-4(*c*), this will tend to increase the resistance to a shearing failure. The weight of the soil above *KL* must be lifted if the earth mass *KHJL* is to move upward and to the right. This principle is very important, especially when one must utilize weak soils. The back pressure due to embedment tends to increase surprisingly the unit pressure that can be applied safely to a granular soil. In fact, moderate pressure tends to make a sand firmer.

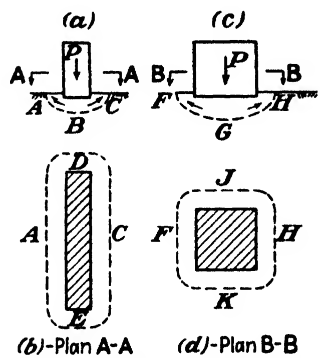


FIG. 4-5. Effect of shape of foundation.

The shape of the loaded area may be important in connection with a possible shearing failure of the soil. Figures 4-5(*a*) and (*b*) show a long narrow structure; (*c*) and (*d*), a square one of equal weight. For purposes of visualization, the shearing action is shown as affecting the surfaces *ABC* and *FGH*, and the earth masses *ADCE* and *FJHK*. The resistances in these two cases may vary considerably. Since plastic soils are the ones that may cause trouble because of shearing and plastic flow, it seems reasonable to assume that the larger perimeter of the narrow structure will result in more area of the surface of possible slippage from under its edges, and, inasmuch as the total cohesive resistance depends partly upon this area rather than upon the weight of the structure alone, there should be a larger resistance to movement under the narrow building than under the square one. Since the magnitude of a frictional resistance depends upon the normal pressure, this frictional resistance should be approximately the same in both cases. The inertia of the mass of earth to be moved in the event of failure should not have much effect in either case because the motion of the soil would be too slow to bring this feature into importance.

It may be possible to excavate a stiff clay and a caliche soil so that the side *AB* of Fig. 4-6(*a*) will stand temporarily in a practically vertical position.

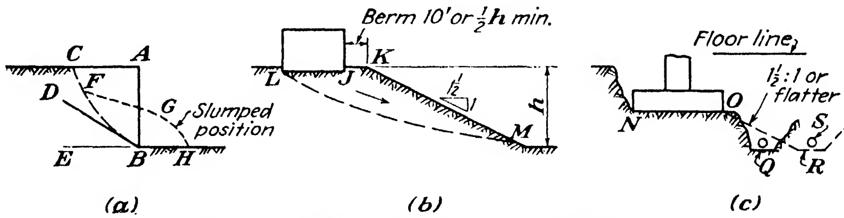


FIG. 4-6. Danger when founding structures near cuts.

tion. However, it may not remain so. Some portion, such as *BCA*, is likely to slide down to *FGHB* because of its own weight, particularly when water and weathering soften the soil. Any extra load placed near *C* will hasten this sliding. A loose granular soil seldom stands up on a very steep slope, not even temporarily.

If a structure must be founded near the top of the slope, the earth should not be steeper than approximately 1½:1, as shown in Sketch (*b*); even then the structure should be set back from the top corner *K*, and the berm *JK* should be rather wide because the lack of support to the right of *KM* will aggravate the tendency for the load to cause slippage along *LM*.

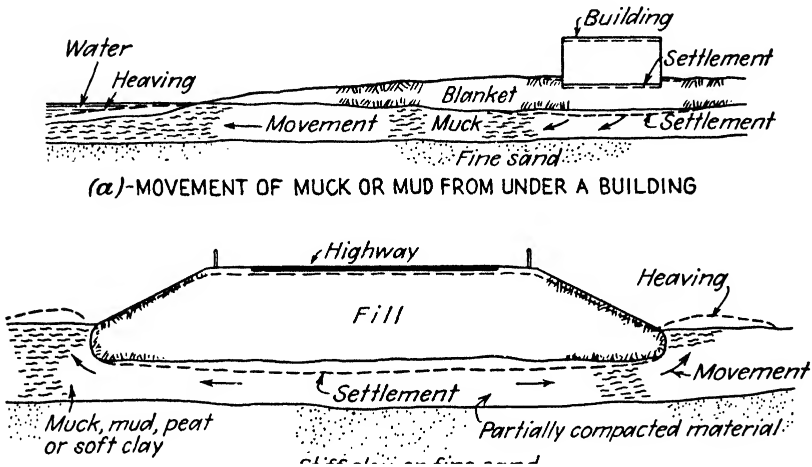
In the case of the footing *NO* of Fig. 4-6(*c*), a trench is to be dug for the drain *Q*. Even though the sides of the trench are shored and the trench is backfilled, the footing *NO* is endangered. The drain should be moved out to some position *S* where the slope of *OR* is not steeper than 1½:1 (preferably flatter), or the footing should be lowered to the level of the bottom of the trench. Conservatism in such matters as these is desirable.

4-4. Settlement. When a structure is built upon earth, some settlement of that structure is practically inevitable. A few general principles to be borne in mind when studying problems of settlement are the following:

1. Granular materials like gravel, coarse sand, and medium sand generally attain their maximum settlement under load as soon as the load is applied, and they retain that settlement; they do not cause the subsidence

to continue to increase over long periods of time. Furthermore, the settlement is usually slight and unimportant.

2. Silt and fine sand attain a large part of their compaction when the load is applied, but the subsidence may increase somewhat over long periods of time because of a decrease of the water content. If the soil is free to move laterally, saturation may expedite its movement under the action of the pressure so that a serious settlement may eventuate. When a real



(a)-MOVEMENT OF MUCK OR MUD FROM UNDER A BUILDING

(b)-MOVEMENT OF PLASTIC SOIL UNDER IMPROPERLY BUILT EMBANKMENT

FIG. 4-7. Examples of possible flow of plastic material from under a load.

quicksand condition develops, the resultant settlement may be disastrous.

3. Clays attain a part of their compaction as the load is applied but, being plastic, they generally continue to consolidate slowly and at a decreasing rate for a long period of time. Most of this is due to a slow squeezing out of the water. A time-settlement curve of such a soil is shown in Fig. 16-6.

4. Muck and mud are so dangerous for supporting loads that a prediction of the settlement of a structure placed on them is largely guesswork. Even when such soils are trapped under a blanket of fill or naturally deposited soil, a small difference of pressure is likely to cause disastrous settlement. These materials sometimes move through great distances and cause subsidence under the load and heaving a few hundred feet away, as illustrated in Fig. 4-7.

5. Temporary loads such as wind, live loads, and traction generally act so briefly that their effect upon cumulative settlements is negligible. In this sense, a week is a brief time. However, a large warehouse for "permanent"

storage is an exception for which it may be desirable to assume a long-term live load of 50 per cent of the design live load when the settlement is estimated.

6. It is generally differential settlement rather than the absolute magnitude of a settlement that causes trouble with a structure. Some illustrations of this are given in Fig. 4-8. When an entire area under and around a building subsides uniformly, when the magnitude of the subsidence is not excessive, and when the settlement does not damage other structures, no one may be the wiser, and the settlement may be unnoticed. However, one is gambling when he trusts this uniformity under conditions in which

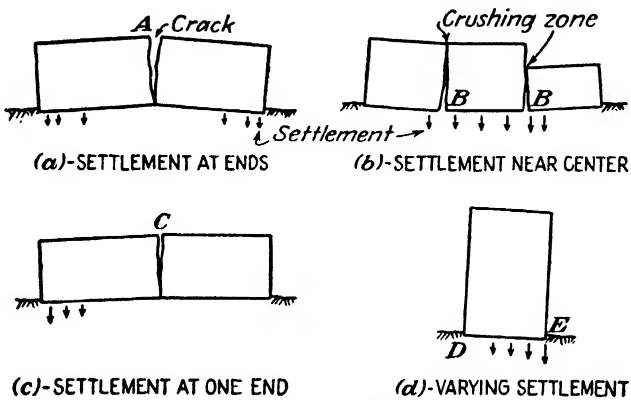


FIG. 4-8. Some harmful effects of uneven settlements.

the settlement is likely to be 6 in., 1 ft., or more. Not only should a building be structurally safe, it should be satisfactory to the owner, useful for the purpose intended, and salable at an unimpaired value. One who has carried on heavy construction work in urban areas will realize the financial loss that owners associate with unsightly cracks, and how valuable are photographs showing the existence of those cracks prior to the start of a construction job when some of these owners file claims for damages.

The following comments refer to the sketches in Fig. 4-8:

(*a*) When the ends of a structure settle more than the center, the building is likely to "break its back," as shown by the crack at *A*. The damage is obvious and may be serious.

(*b*) When the ends settle less than the center, the top of the building is compressed. This is not likely to cause serious trouble unless the deformations are so great that horizontal shearing ruptures the structure, or tension cracks its bottom.

(*c*) Settlement of one end of a long building with respect to the other end may crack the superstructure near the top, as at *C*.

(*d*) Gradually increasing settlement from one side of a tall building to the other may tilt the entire structure without cracking it. This may be psychologically if not structurally harmful.

7. Frost action may cause local heaving, subsequent softening of the soil, and localized settlement. Therefore, the bottoms of foundations should be placed below the probable frost line. This depth may be 5 to 6 ft. in northern New York, Maine, and Minnesota; 4½ to 5 ft. in slightly warmer climates; about 4 ft. in New York City; and approximately 3 ft. in somewhat milder regions. Because of the desirability of proper embedment and the removal of topsoil, no important foundation should be less than approximately 3 to 4 ft. below the ground surface, even in warm climates. Interior column footings of structures in the North that may be unoccupied and unheated for long periods during the winter should also have reasonable embedment.

Under refrigerated warehouses and cold-storage places, the foundation should be protected from freezing. The cold from subfreezing temperatures penetrates to great depths when it continues for long periods, and the formation of ice lenses may cause heaving and the presence of tremendous forces. Then, when refrigeration is stopped for a considerable time, thawing may cause critical settlements. At the edges of the foundation, deep freezing of the soil may occur in the winter, and then warm ground water may thaw the soil under the main walls, whereas it cannot penetrate rapidly to the center of the foundation. This action may enable the force of gravity to pull down the outer shell of a building around the rigid frozen core, thus cracking the structure seriously.

Insulation of a soil-supported floor to prevent the transmission of heat (or cold) may be utterly ineffective. Insulation retards the transmission of heat, but it cannot stop conduction entirely. When frigid temperatures exist continuously on top of the ground floor and when there is no way of dissipating the cold, the latter will penetrate slowly in spite of insulation. A ventilated basement, a network of closely spaced ducts through which sufficiently warm air is circulated, a system of piers to hold the insulated floor off the ground and permit natural circulation under it, and the use of the basement or ground floor for the storage of fruits and vegetables at temperatures considerably above 32°F. are all means whereby this problem of freezing under such warehouses can be conquered.

8. A compressible stratum below a much firmer one may still constitute a hazard and cause serious long-term settlements. An estimate of probable settlement should take into account the soil strata that are within some reasonable distance of the bottom of the substructure. The suggestions in Table 4-1 may be helpful in determining the depth below which the settlement caused by the weight of the structure may be assumed to be unimportant.

TABLE 4-1. Minimum Average Unit Pressures to Consider When Estimating Probable Settlements

Character of soil	Average pressure p, p.s.f.
Deep soft clay	300
Thin stratum of soft clay between firm impervious strata	500
Deep stiff clay	500
Thin stratum of stiff clay between firm impervious strata	700
Deep silt and very fine sand	500

In the table, p denotes the average unit pressure caused by the structure, assuming a 2:1 distribution. The higher unit pressures for thin strata—10 ft. thick or less—when trapped between firm impervious strata are suggested because the water in the plastic layer cannot escape easily when the pressure is applied; on the other hand, a porous soil will not greatly retard the movement of the water that is squeezed out of the neighboring stratum.

9. Consolidation tests of plastic and weak soils should be made in a soil-mechanics laboratory upon some such basis as the following:

a. One test for each important stratum that has, or seems to have, considerably different constituents, grading, or properties than the stratum directly above it. These tests are to be made on undisturbed samples taken from at least one selected boring.

b. One test for each 10 to 20 ft. of depth of thick strata. The number of tests to be made will depend upon the judgment of the engineer when he considers the importance of the case and the properties of the soils as already partly revealed.

c. When the borings indicate that the soil varies considerably in character over the site, tests as in (*a*) and (*b*) should be made on the soil samples from some one of the borings in each area where important variations are apparent.

d. If the results of the soil tests show the existence of dangerous or unsatisfactory materials, additional tests on undisturbed samples from other borings should be made as the situation seems to warrant.

It is advisable to prepare load-settlement curves from the laboratory tests, especially when the seriousness of the probable settlement makes it desirable to have such data for use in studying alternate schemes for the support of the structure so that the one adopted may be the most economical plan consistent with safety and permissible settlement. One should remember again that settlements may be limited to small magnitudes but at unjustifiable expense; on the other hand, harmfully large settlements that ruin or seriously damage a structure may mean, in effect, that the entire expenditure for the structure has been wasted.

Assume that the site of an important structure is being explored and that four undisturbed samples have been taken from the first boring. These have been rushed to a laboratory for preliminary tests to ascertain whether the consolidation and settlement will be serious, and to serve as a guide for further exploration and sampling. Assume also that the samples are being tested at loads of 1 and 2 tons per ft.^{2} only.

Perhaps part of the initial report of the consolidation tests is the following:

Boring No. 1, Sample No. 2 : Stiff Blue Clay

Unit pressure, p.s.f.	*Voids ratio, e*
0	0.886
2,000	0.864
4,000	0.849

These data may be used to plot a curve of unit pressure vs. settlement by using the equation:

$$\Delta h = \left(\frac{e_i - e_p}{1 + e_i}\right) h \tag{4-3}$$

where Δh = total expected settlement under load in inches, e_i = initial voids ratio, e_p = voids ratio when under a pressure p, and h = thickness of layer in inches.

It may be convenient to draw a diagram to show at a glance the estimated settlement in inches for a stratum 10 ft. thick when a particular unit pressure is applied to it, as judged from the results of the tests. For this sample,

$$\Delta h_{2,000} = \left(\frac{0.886 - 0.864}{1 + 0.886}\right) 10 \times 12 = 1.4 \text{ in.}$$

and

$$\Delta h_{4,000} = \left(\frac{0.886 - 0.849}{1 + 0.886}\right) 10 \times 12 = 2.35 \text{ in.}$$

The curve for this case is plotted as No. 2 in Fig. 4-8*A*. The diagram should be curved, but straight lines between the reported readings are used here for samples 1 to 4, since the diagram is prepared only to enable one quickly to obtain a scale on the expected settlements. Further tests and curves can be made later for use in checking the final design. However, the methods of computation shown here are for preliminary investigations. Since the 2:1 distribution is only an assumption, the data of Fig. 4-8*A* seem to be suitable for use therewith. Later on, the use of the Newmark circle of Fig. 4-3*A* and curves made from the final laboratory tests may be used to make the final estimates of settlement after the structure has been planned and the loads and bearing areas are more fully determined. The general method of summation is similar to that for the preliminary estimates.

It is hazardous to attempt to show load-settlement curves for a typical clay or silt because they vary widely. Some organic clays may have voids ratios equal to 2 or 3, and they may compress to yield voids ratios that are only 50 to 75 per cent of these under 3 or 4 tons per ft.², if they do not flow away in the meantime. Clays that have been compressed for ages under a much heavier overburden than now exists (as under glacial pressures) may be quite dense and reliable. Test results made on one sample of clay are shown in *a* of Fig. 4-8*A*, and those on a silt are given in *b*. Perhaps these are medium to soft materials.

Figure 4-8*A* indicates that sample No. 1, which was taken near the surface, was more compressible than the deeper materials. This may or

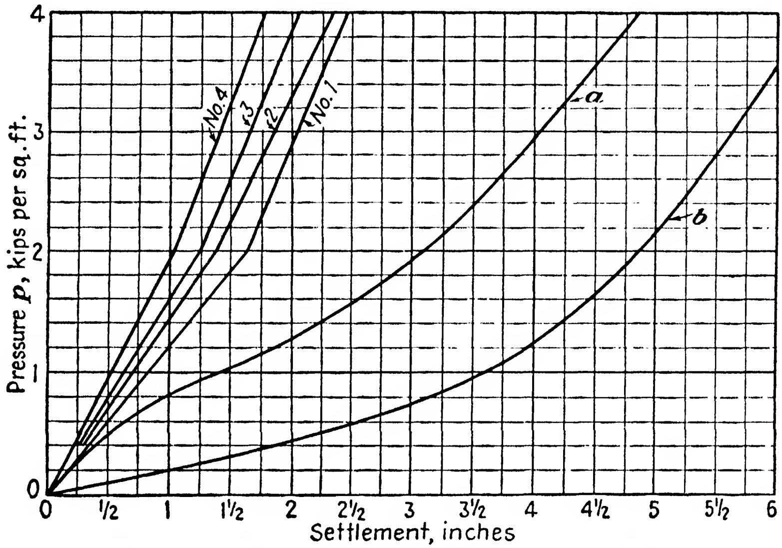


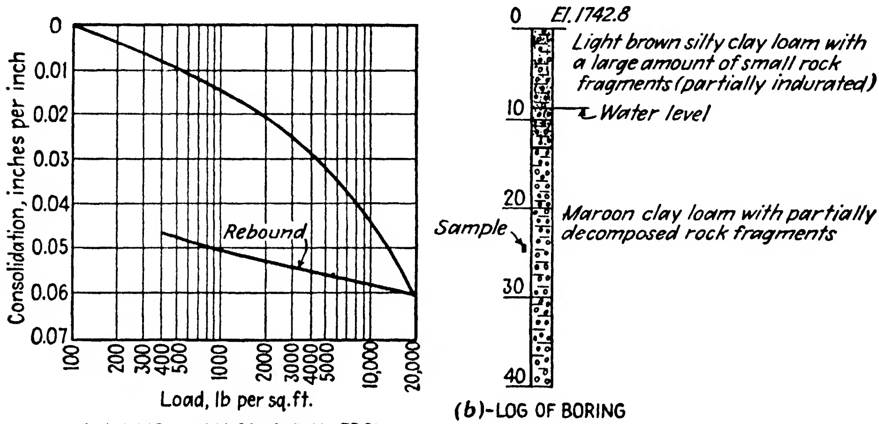
FIG. 4-8A. Load-settlement curves for a 10-ft. stratum of soil. Such curves are to be plotted from results of laboratory tests on undisturbed soil samples.

may not be the case in nature. It is shown this way purposely in order to illustrate later on the use of such curves when preliminary estimates of settlement are made.

Another question in connection with the use of the laboratory tests showing load-consolidation relationships is what to allow for the effect of the weight of the overburden. By necessity, a soil sample is removed from pressure when it is taken out of the ground. How much does this change it? How much is it remolded? How different will it behave when in place? The effect of *added* load on the soil in place should be known.

Undoubtedly, there will be a slight rebound when pressure is removed from the sample, but its amount is seldom important. The voids ratio will not suddenly jump up. Good sands need not be a matter of great concern because pressure usually makes them firmer. Silts generally are consolidated to some extent under the pressure of the overburden, and they will not "deconsolidate" unless disturbed too much. The same applies to clays. However, some saturated clays may not be consolidated very well because much of the pressure has been withstood by the water "trapped" in the pores. Soft plastic clays may be under many feet of overburden, as shown in Fig. 16-2.

In general, it seems that the load-settlement curves as determined by the tests can be used safely in estimating settlements caused by superimposed loads, except for the time element. This omits the weight of the overburden as such from the computations. The compaction of the samples



(a)-LOAD vs. CONSOLIDATION FROM CONFINED COMPRESSION TEST

(b)-LOG OF BORING

FIG. 4-8B. Report of consolidation test on undisturbed soil sample taken 24 ft. below surface in Arizona. (After report by Dames and Moore.)

may generally be somewhat larger than that of the soil in place under pressure, hence computed results tend toward the side of safety.

Of necessity, these laboratory tests are of relatively short duration. The element of time should be considered when investigating clays because it is obvious that, in an actual case, the moisture will continue to be squeezed out for a period of years. If the pressure is large and the original voids ratio is large, the situation is more dangerous than if one of these factors is relatively small. The results of one's computations of settlement should be modified according to the factor "J." Again one should remember that, once the structure is built, it is probably impossible to change the product of the basic assumptions used in the planning. The performance of the structure is real, not imaginary.

When problems of settlement are being studied, it is often helpful to try to visualize what the unit pressures mean in terms of the weight of some physical object. For this purpose, an imaginary pile or block of earth that will produce the same unit pressure may be pictured as resting upon the soil. Therefore, if one plans a unit pressure of 4 tons per ft.[2] under a footing, this is somewhat equivalent to the weight of a column of earth 80 ft. high, approximately the height of a seven- or eight-story building. This may be entirely satisfactory on sand and gravel. but not on a soft clay.

Perhaps it would be helpful to try to visualize this consolidation of soil. Figure 4-9 has been prepared for this purpose.

A granular soil generally has a considerable volume of voids throughout the mass even though it is fairly well graded. Such a soil is pictured to exaggerated scale in Fig. 4-9(*a*). When a load is placed on this soil, the

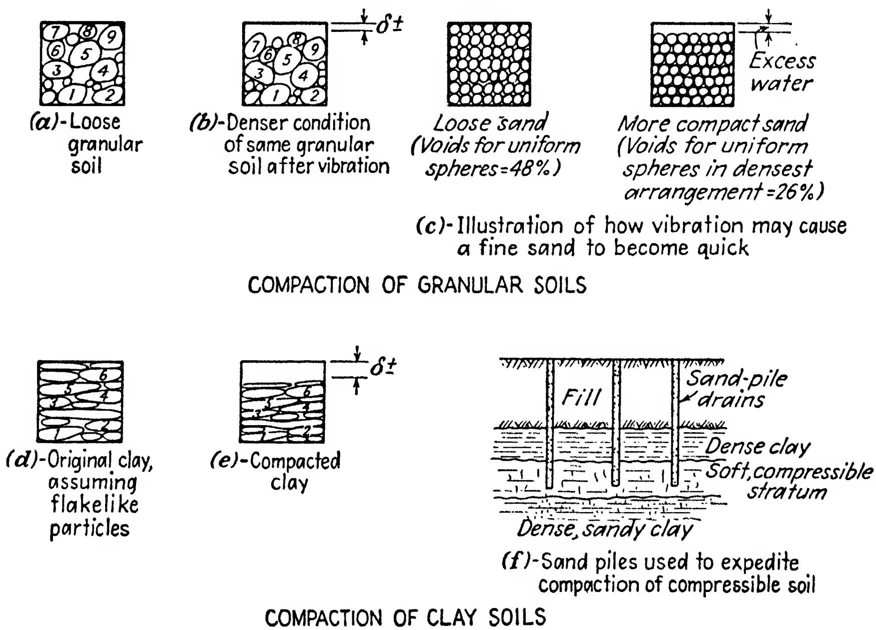


FIG. 4-9. Illustration of compaction of soils.

pressure is transmitted by the bearing of grain on grain, and only slight deformation is likely to occur unless the particles are crushed. However, the loads are seldom such as to cause a mashing down of a siliceous sand or gravel, although they may do so in a material composed of flaky weak shale.

On the other hand, strong vibrations may cause particles to slip into voids or to readjust themselves somewhat as pictured in Fig. 4-9(*b*). This increases the density of the mass and, since the total solid matter remains the same, the reduction of volume usually causes a vertical settlement. Practically, this action is not often serious except when the material is saturated fine sand, or possibly a silt. Any sudden disturbance that "shakes the solids down" automatically "flushes water out," as illustrated in Fig. 4-9(*c*). This water may then tend to lubricate or "float" very fine sands and form a flowable material, at least temporarily. The sand may then become quick, flow from under the load, and cause sudden disaster. On the other hand, if flow of the solids is prevented, as by enclosing the loaded area within suitable sheet piling, there can be no serious settlement beyond that caused by the displacement of the excess water as the sand becomes denser. Of course, the driving of piles may produce the vibration that will shake down the grains.

In one case, an addition was to be built alongside an old brick factory in New England. It was supported on wall footings resting on loose glacial sand. The new portion was to have a basement extending several feet below the old footings. However, the basement wall was to be kept 10 ft. away from the old footings. Steel sheet piling was to be driven outside the old wall to hold the soil. Contrary to orders, the contractor drove this piling heavily with a drop hammer. One corner of the old building settled approximately 1 in. and opened up an ugly crack.

The ordinary soils that may cause settlement troubles are clays, sands or silts mixed with considerable clay, and sometimes silts alone. Figure 4-9(*d*) is an exaggerated picture of the assumed particles in a clay. The individual particle is supposed to be weak structurally, to be flakelike, and to be somewhat spongy. The mass is supposed to be in equilibrium and to be saturated, or nearly so. When a load is applied, the "skeleton" of the mass may break down in places, the particles rupture or distort, and the solid material become packed more closely, as shown in (*e*). The water, being incompressible, will be squeezed out and will flow away, if it can, somewhat as when one steps on a wet sponge.

Since clay is rather impermeable, it may require a long time for this water to escape completely, and the progress of settlement will be slow but prolonged. If the clay is trapped within other dense strong impermeable materials, escape of the water and appreciable settlement may be prevented. However, it is often dangerous to trust this because of possible disturbance or change in the future. An adjacent layer of sand or gravel, on the other hand, may be like pipes in conducting the water away. In fact, holes are sometimes made by driving a pipe pile down to a compressible material, as shown in Fig. 4-9(*f*), and filling it with sand; then the pipe is withdrawn. These "sand piles" expedite the escape of the water (and therefore facilitate the compaction) of a compressible stratum. They are useful when a heavy fill is spread over soft marshy ground.

The total long-term settlement that will occur because of such consolidation is difficult to predict. Consolidation tests made in the laboratory will give some information. When the conditions appear to be dangerous, it is better to plan the structure so that one plays safe rather than to take chances and be sorry in the future.

Occasionally, one encounters something special. At a plant in northern Mexico, it was planned to build a bucket elevator structure about 20 ft. square and 115 ft. high. Adjoining structures were under construction.

When the excavation for the elevator house was started, the soil was found to be a slightly moist spongy disintegrated volcanic material, a sort of pothole 50 ft. deep being filled with it. Walking on it felt somewhat like walking on a thick layer of pine needles in a forest. Relocation was inadvisable; no piles or pile-driving equipment was at hand; deep excavation and construction would be costly. Therefore, to avoid danger, the order was canceled, pumps were substituted to handle the muddy liquids, and a low pump house on a mat was built to "float" on the spongy mass. The loads were so light and widely distributed that the settlement was negligible.

Still another problem developed at a plant in Chile where large dewatering tanks were to be built on desert soils, mostly caliche. Samples showed about 15 per cent of the material to be soluble in water. Would leakage of water from the tanks remove the solubles, with consequent subsidence of these costly structures? It was thought that the soils were probably deposited and consolidated ages ago and that, subsequently, the chemicals were deposited in the voids, largely as a result of evaporation. If so, solution would remove them but would not cause a settlement of the soils. Special tests, devised to prove this idea because of the tremendous sums involved, showed the generally anticipated loss of weight but no loss of volume when the chemicals were leached out, not even when the leached samples were vibrated by hitting the container with a hammer. Therefore harmful settlement was not likely.

These cases are given to show again how foundation problems vary. Each is an individual problem, and the best solution should be found for the special conditions as they are and where they are.

4-5. Estimates of settlement. *1. Settlement of a structure on a thick plastic soil.* Assume that the structure shown in Fig. 4-10(a) is to have a heavy concrete mat at its base, and that the unit pressure at AB is to be 2 tons per ft.^{2} The soil is a 40-ft. layer of clay above a thick stratum of sand. Assume further that undisturbed soil samples were taken at 10-ft. intervals, as indicated, and that the pressure-settlement curves for a 10-ft. depth as determined by laboratory tests are as pictured in Fig. 4-8A. How much settlement may be expected?

For convenience, the 2:1 distribution will be assumed. The estimate will be made in three ways for purposes of comparison.

a. Use of average of unit pressures at top and bottom of stratum, found as follows:

$$P = 50 \times 50 \times 4 = 10{,}000 \text{ kips}$$

$$p_{AB} = 4 \text{ k.s.f.}$$

$$p_{JK} = \frac{10{,}000}{90^2} = 1.2 \text{ k.s.f.}$$

$$\text{Average } p = \frac{4 + 1.2}{2} = 2.6 \text{ k.s.f.}$$

Average settlement Δh for 10 ft., interpolated for $p = 2.6$ from the mean of the curves for samples No. 2 and 3 of Fig. 4-8A, is $1\frac{5}{8}$ in. Therefore, the approximate total $\Delta h = 4 \times 1\frac{5}{8} = 6\frac{1}{2}$ in.

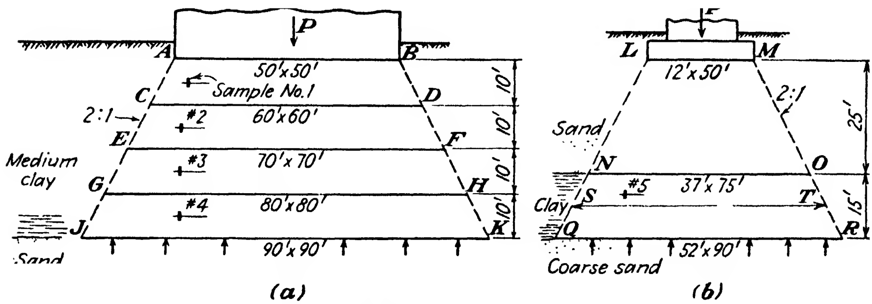


FIG. 4-10. Procedure for roughly estimating settlements.

b. Use of unit pressure at middle of stratum:

$$p_{EF} = \frac{10{,}000}{70^2} = 2 \text{ k.s.f.}$$

Average Δh for 10 ft. determined as in (*a*) is $1\frac{3}{8}$ in. Therefore, the approximate total $\Delta h = 4 \times 1\frac{3}{8} = 5\frac{1}{2}$ in.

c. Use of a series of imaginary 10-ft. layers:

In this case the 40-ft. depth is divided into 10-ft. layers. The computations are tabulated as follows, with values of the settlement for each layer scaled from the proper curve in Fig. 4-8A:

Stratum	Sample No.	p, k.s.f.	Δh, in. per 10-ft. depth
AB-CD	1	$10{,}000/55^2 = 3.3$	$2\frac{1}{8}$
CD-EF	2	$10{,}000/65^2 = 2.4$	$1\frac{5}{8}$
EF-GH	3	$10{,}000/75^2 = 1.8$	$1\frac{1}{8}$
GH-JK	4	$10{,}000/85^2 = 1.4$	$\frac{3}{4}$
			$\Sigma\Delta h = 5\frac{5}{8}$

All these results are estimates only. The computations need not be carried out to more than two significant figures because even the first one is not likely to be "correct." The "undisturbed" samples on which the tests are made are remolded somewhat so that they do not exactly duplicate the properties of the soil in its natural state. The computations do, however, show whether the settlement is likely to be 6, 2, or 12 in. In many cases, method (*b*) is sufficient for one's purposes.

Now look at the results of these computations. If the structure settles this estimated amount, it is probable that the surrounding soil will subside also and form a saucerlike depression with the structure at its center. This may cause no serious difficulties, but a settlement of this magnitude is not satisfactory to the owner or the engineer. If railroad tracks, overhead bridges, flues, adjacent buildings, and other structural features are alongside or connected into the building, they may advertise the settlement to all observers. Furthermore, it is probable that the central portion of the assumed loaded mass of earth will settle more than that estimated for the average pressures, perhaps by 30 to 40 per cent. Will this tendency of the mat to sag in the center cause harm to the structure because of stresses or deformations?

Because of a tendency to sag near the center, the structure should be designed as a strong box or block that will retain its desired shape. Then the rigidity of the structure may cause the pressure to equalize more fully as the plasticity of the soil produces a readjustment of the latter. However, because of the long-term loading and the gradual compaction of the clay, the eventual settlement of this structure may be as much as 8 in.

In this case, it is desirable to study some such remedies as these:

a. Make the structure considerably lighter.

b. Spread the load over a much larger area. This may require a large low structure rather than a narrow heavy one.

c. Change the type of foundation by using piles to reach into the sand stratum.

d. Look for a more favorable location.

2. Settlement of a structure on sand with an underlying plastic stratum. Assume that Fig. 4-10(*b*) represents the base of a high bridge pier near the edge of a sluggish stream. Can the pier be supported safely upon the sand when a 15-ft. layer of clay underlies it?

Assume that the load to be on the base LM will cause an average pressure of 3 tons per ft.^{2} One undisturbed sample, No. 5 in the picture, was taken from the clay stratum. Assume that the clay's properties are similar to No. 2 in Fig. 4-8*A*, and use the 2:1 ratio for distribution of the pressure.

The settlement of the 25-ft. stratum of sand will not be important by itself. Hence the compaction of the clay alone is to be estimated. It is satisfactory to use the average unit pressure at the center of the clay stratum and, with this pressure, find the settlement of a 10-ft. layer from No. 2 in Fig. 4-8*A*, and multiply this by 1.5 because the layer is 15 ft. thick. The resultant computations are the following:

$$P = 12 \times 50 \times 6 = 3{,}600 \text{ kips}$$

$$p_{ST} = \frac{3{,}600}{44.5 \times 82.5} = 1.0 \text{ k.s.f.}$$

$$\Delta h = \tfrac{3}{4} \times 1.5 = 1\tfrac{1}{8} \text{ in.}$$

This settlement might be acceptable if the superstructure is of a type that is not harmed by such a subsidence; *e.g.*, simply supported steel or concrete girders and steel trusses, and (possibly) long shallow steel girders designed to withstand the deformations caused by the settlement of one pier. However, some arches and stiff continuous structures are likely to be endangered by such settlements.

A decision of acceptance or rejection is difficult to make, and one must abide by it after it is made because he cannot change his mind after the pier is built. A steel mill building might stand such a differential settlement, and human lives might not be endangered. A large bridge project that is to support heavy traffic is a more serious matter. Furthermore, the computed settlement is an estimate only. If all the piers of the bridge are founded similarly and if the estimated settlements are practically equal, it may be safe to accept such foundations.

In this case, how large would the base LM, Fig. 4-10(*b*), have to be in order to reduce the estimated settlement to $\frac{1}{2}$ in.? From curve No. 2 of Fig. 4-8*A*, for a settlement of approximately $\frac{3}{8}$ in. in the 10-ft. stratum, $p = 0.5$ kip. Therefore, the area of the central plane of the clay layer would have to be $3{,}600/0.5 = 7{,}200$ ft.² The new dimensions of the footing should then be increased to something like the following, where $x =$ the new width and $x + (50 - 12) =$ the new length:

$$(x + 32.5)(x + 38 + 32.5) = 7{,}200$$
$$x = 35.5 \text{ ft.} \qquad x + 38 = 73.5 \text{ ft.}$$

This size is obviously impractical, and it shows that minor increases in the area of a footing have little effect upon settlements caused by deep-lying and thick strata.

Many an engineer has founded structures upon plastic materials and been gratified when they proved to be successful, but many another has wished that he had founded them more securely.

3. Settlement of closely spaced footings on plastic soils. Figure 4-11 pictures the proposed two-column pier of a highway viaduct or bridge approach. The unit pressures under AB and CD are to be 3 tons per ft.² How much settlement may be expected?

In this case, the inner assumed 2:1 distribution lines intersect at K. Therefore, the assumed areas of resisting pressure at any lower level abut at the line KY. The arbitrarily chosen zones of pressure are divided into three layers 10 ft. deep. The average unit pressure at $XY = (100 \times 6)/(30 \times 40) = 0.5$ kip per ft.²; therefore, settlement of the soil below this level will be disregarded. Because soil samples were taken only at the locations shown, the settlements are computed for 10-ft. layers, assuming the same soil characteristics as for samples No. 2, 3, and 4 of Fig. 4-8*A*. The computations are as follows, using $P = 600$ kips:

Stratum	Sample No.	p, k.s.f.	Δh, in. per 10-ft. depth
AB-JK	2	$600/15^2 = 2.7$	$1\frac{3}{4}$
JK-RS	3	$600/(22.5 \times 25) = 1.1$	$\frac{5}{8}$
RS-XY	4	$600/(27.5 \times 35) = 0.6$	$\frac{3}{8}$
			$\Sigma\Delta h = 2\frac{3}{4}$

This settlement seems to be altogether too much for a viaduct pier. One can see that making the footing continuous from A to D would reduce the unit pressure directly under the footing, but it would have little effect upon the total settlement because practically the same volume of soil must sup-

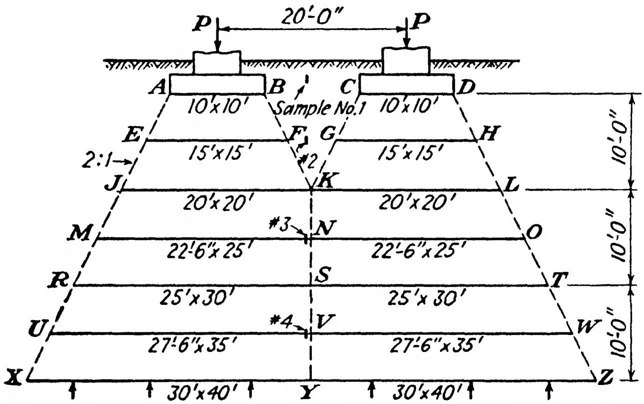


FIG. 4-11. Estimating settlements when "pressure pyramids" of distribution overlap.

port the structure in both cases. Such a continuous footing, however, or a strut between B and C, might be useful because inevitably higher pressures in the vicinity of KY might cause greater subsidence of the center, with the possibility of tipping of the footings toward B and C. It is also obvious that, for small footings, the assumption of rectangular pyramids of distribution of pressure yields appreciably smaller estimated settlements than would circular cones of distribution. Here again, other designs should be studied before one accepts so large a settlement.

4-6. Structures founded upon fill. As a general rule, it is dangerous to support a structure upon filled ground. On the other hand, it might be desirable from the standpoint of location and relation to other structures. If one must make a decision regarding such a question, he should study the problem very carefully.

Fills should be made of granular soils, although a little clay binder may be helpful. The use of silt and very fine sand is not advisable if better materials are obtainable. Fills should be constructed in thin horizontal layers that are compacted thoroughly as each one is made, using sheep's-foot rollers or other acceptable equipment. Fills that are made—or have been made—by dumping dirt from a truck down the face of a slope of previously deposited fill will be poorly compacted, uneven in density, partly segregated, and more or less stratified along inclined planes; hence they are dangerous as foundation materials. Sprinkling the soil with water as the horizontal layers are placed often helps to wash the fine particles into the interstices between the larger ones, thus making a denser fill. In spite of one's best efforts, some future settlement of a fill is almost unavoidable, but it can be reduced acceptably for some purposes. Time and weathering, plus the compaction caused by its own weight, will generally produce a progressively decreasing annual settlement of the fill itself. Perhaps 4 or 5 years will be necessary before an important structure can be supported safely upon such filled ground. However, one cannot expect to dig out a soil, transport it, and recompact it in another location as well as nature did at the original site throughout long periods of time. This is understandable when one realizes that the bulking of coarse sand when excavated may be 3 to 5 per cent, whereas that of moist fine sand may be 10 per cent. The original density cannot be restored easily.

An old existing fill may or may not be trustworthy. It is easy to say that a fill should be investigated carefully before it is utilized. It is another thing to do this thoroughly and yet economically. It is probable that an old fill was made by dumping the soil down a slope from trucks or wagons, and it was never intended to support important structures. The material in the fill may be almost anything from junk to rocks.

In one case, a fill 10 or 12 ft. deep had been made near a river's edge about 40 years ago. The borings indicated that the fill was made of sand, gravel, old bricks, and miscellaneous earthy materials, and that it was underlain by medium clay, silt, and finally by sand. A small but important industrial building was to be erected at that spot. It was decided therefore to use a bearing pressure of less than 1 ton per ft.^{2}, and to found the structure on the fill about 4 ft. below the ground's surface rather than to spend many thousands of dollars driving piles to the lower sand stratum. When the excavation was made, it was found that a mass of old wooden shavings was encountered below the watertable near one corner of the substructure. These were removed, and the space was filled with lean concrete. The work was allowed to proceed because it seemed too late and otherwise inadvisable to alter the contract because of this discovery. However, one cannot help worrying about what else may be hidden below the structure.

Referring to Fig. 4-12, there are two principles illustrated which the reader should notice. The situation of the shavings is pictured in Sketch (*a*). In (*b*), the earth is shown excavated with a flat area at AB and a very moderate slope BC, with the lean concrete fill deposited first so that the reinforcement of the mat may be placed above a level surface. If the side BC were made steep, as DE in Sketch (c), the bearing power of the soil at EF might be weakened seriously. If the lens of shavings had been farther in from the edge, as indicated in (d) and if the concrete fill were placed as shown, the slope GH should not be used because it would aggravate the tendency of the soil to squeeze out from under the footing. One would be wiser to make the bottom flat, as shown by JH. These basic principles apply to any earth foundation as well as to the use of fill.

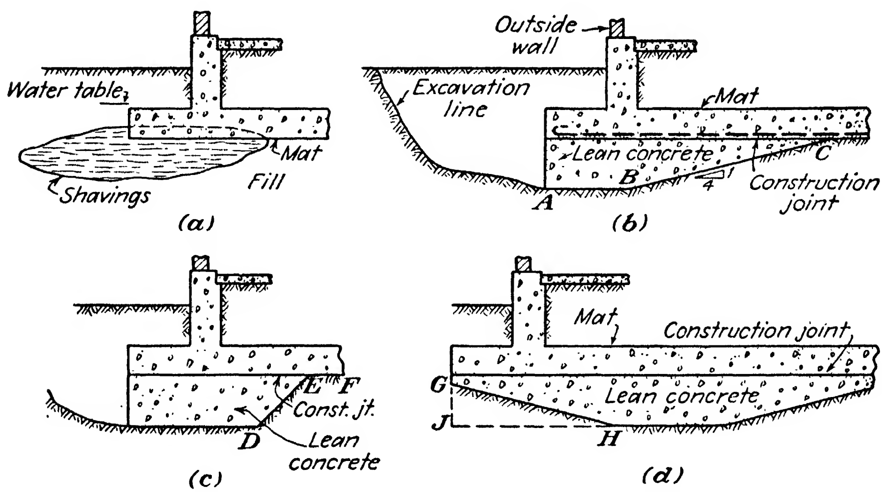


FIG. 4-12. Filling of weak spot under a substructure.

When a building must be supported upon fill, it is generally advisable to design it so that the settlement will not endanger it. Of course, the structure should be as light as practicable. The frame should be sufficiently flexible to permit uneven settlements if they are probable; a series of stress-relieving joints might be used through the substructure and the superstructure; or the building might be made strong enough to act as an entity without cracking.

Figures 4-13 to 4-15 show the principles used in the design of some structures when new or old fill was to be utilized. Each case was a matter for individual study, and the illustrations are simplified in order to show the principles more clearly. The ideas are also applicable when one encounters strata of weak plastic soils of similar dimensions and shapes as the fills pictured. The following comments are given in connection with the respective illustrations:

Figure 4-13. A large machine shop was desired in a particular location in an industrial plant, this location being determined largely by the available space and the problem of getting railroad tracks into the structure. An area of approximately 60 × 75 ft. at one corner would be over the head end of a small valley so that the floor of this end would have below it a wedge-shaped space that would vary from zero to 18 ft. in depth, somewhat as pictured in the sketch. If this area were filled in and if the building were supported directly upon it, the settlement of this corner might damage the structure and, especially, harm the crane runways and the machinery foundations. Therefore, concrete piers with spread footings were erected at all columns and under heavy machines, the fill was placed between and around these piers, and a reinforced-concrete beam-and-girder floor was laid directly on top of this fill. The floor was seated on the piers and was

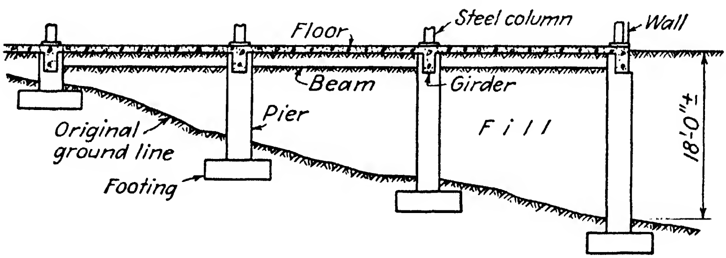


FIG. 4-13. Reinforced-concrete floor supported upon concrete piers.

designed to carry all the loads so that, when the fill settled, the floor would not be affected.

It is very difficult to compact fill thoroughly when it is placed between an assembly of these piers. The fill may be placed first, then piles can be driven through it if it contains no large gravel and boulders. This, however, is costly, and the piles will have poor bearing value unless they are driven well into the original ground. Another method is to sink pipes through the fill, excavate inside them, fill them with concrete, and withdraw the pipes. On the whole, it seems preferable, when the fill is not to be used for supporting loads, to construct simple piers in 10-ft. lifts, place the fill approximately to their tops, and then proceed with the next lift. One should be careful to avoid unbalanced filling that will tip the piers.

Figure 4-14. This small warehouse in Arizona was built upon new fill having an approximately uniform depth of 30 ft. The original ground was good. The structure did not seem to warrant the expenditure necessary for tall piers or piles. Furthermore, it was not desirable to have the adjacent railroad tracks settle with the fill, whereas the warehouse remained stationary. It was therefore decided to rest the structure upon the fill, but to make it so that it would not be damaged if settlement occurred.

The basement under one end of the building made it probable that this end —with less fill under it—would not settle so much as the remainder. On this account, a stress-relieving joint was made through the entire structure at A, Sketch (a). The shallow part was seated upon a shelf on the basement wall, as illustrated by points B, so that the two portions could not shear apart. The basement was naturally a stiff boxlike structure. The section AC was designed somewhat like an inverted boat that floated upon the fill. The longitudinal and end walls D and E served as stiffening ribs;

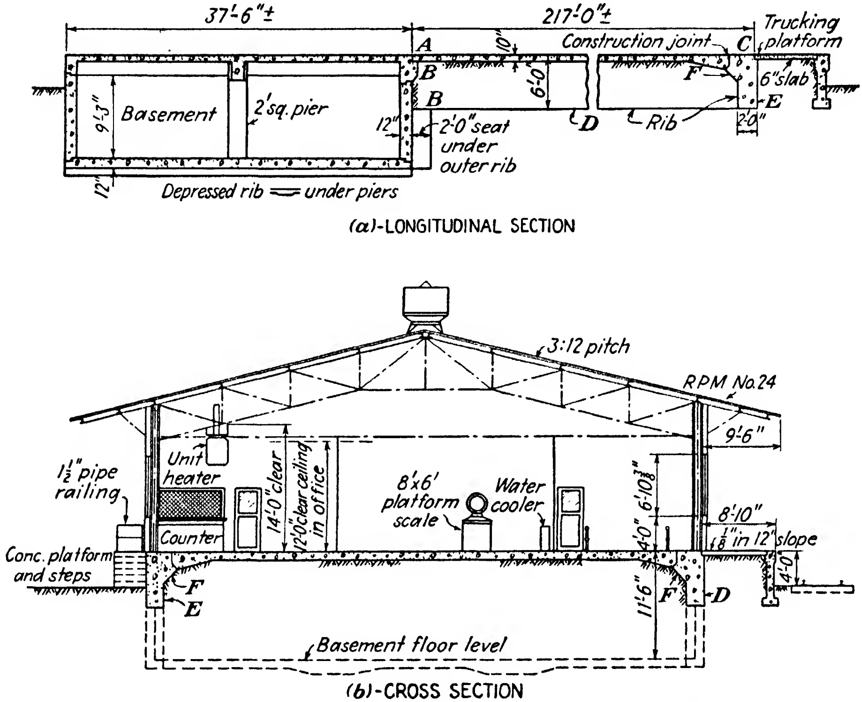


FIG. 4-14. A structure with a mat foundation on a fill 30-ft. deep.

the fillets at F enabled the floor to act as a continuous part of these ribs and resist upward or downward forces.

The fill, a mixture of gravel and sand, was placed with great skill. No noticeable subsidence has occurred because the building has remained intact, and the entire area has behaved uniformly. Even a brick-walled one-story change house built nearby, on the same material and of the same basic design, shows no cracking.

In another case, a one-story building approximately 50 by 180 ft. was built upon a slightly tapered sandy gravel fill. The structure had brick walls, reinforced-brick pilasters that acted as columns, steel roof trusses, a suspended ceiling, and interior brick or hollow-tile and plaster partitions. The substructure was made similar to Sketch (b) except that the floor was only 6 in. above the surface of the fill so that the side stiffener walls were not so deep. Because of the localized weight of the partitions, cross ribs were used under some of them, approximately 20 to 25 ft. apart, in order to stiffen the structure transversely. No trouble has occurred with the structure in the 6 years of its life.

Figure 4-15. This pictures a deep fill used as the approach to the steel viaduct at the western end of the approach to the Lincoln Tunnel, over the New Jersey Meadows, where connecting roadways converge at the main structure. The fill was placed on top of a thick stratum of clay. It was expected that the fill would compact and that the clay would do likewise when such a tremendous load was placed upon it. The steel structure was

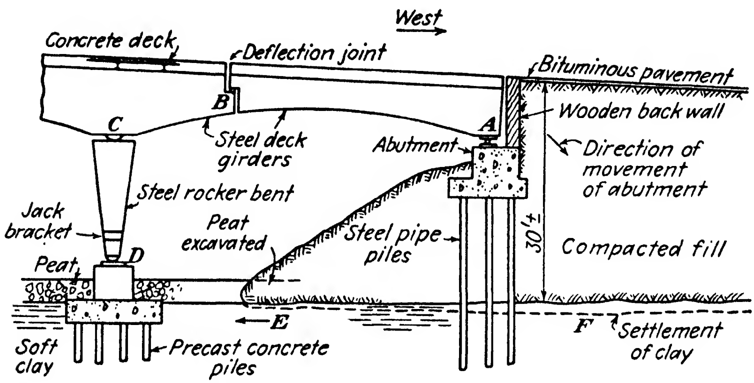


FIG. 4-15. Simplified illustration of viaduct structure and fill at western end of the New Jersey Approach to the Lincoln Tunnel in New York City.

therefore supported upon a roller *A* which would permit considerable longitudinal motion; the deck girders were made with a rocker at *B* to allow vertical movement of *A* with respect to *B*; and the legs of the adjoining steel bent were made with pivoted supports at *C* and *D* in order to permit lateral motion of *D* with respect to *C* if the clay moved out from under the toe of the fill, as indicated at *E*.

The abutment itself was a temporary structure, partly of wood, with a concrete mat on steel tubular concrete-filled piles whose purpose was to eliminate from the vertical settlement of the abutment any effect of compaction of the fill itself. Considerable compaction of the clay was expected, and it was realized that the piles would be carried along with any movement of the clay in which they were embedded. In 2 years, the abutment settled downward about 16 in. and toward the west so that the contemplated adjustments had to be made. The clay seemingly compacted more at *F* than at *E*. Furthermore, the pier *D* moved eastward slightly, as expected, so that the bent was jacked up and the bearing *D* was adjusted.

4-7. Danger signals. Rock, gravel, and coarse sand are good materials for most foundation purposes, but irregular surfaces of these firm materials with varying depths of plastic soils over them may constitute dangerous conditions from the standpoint of unequal consolidation of the overlying

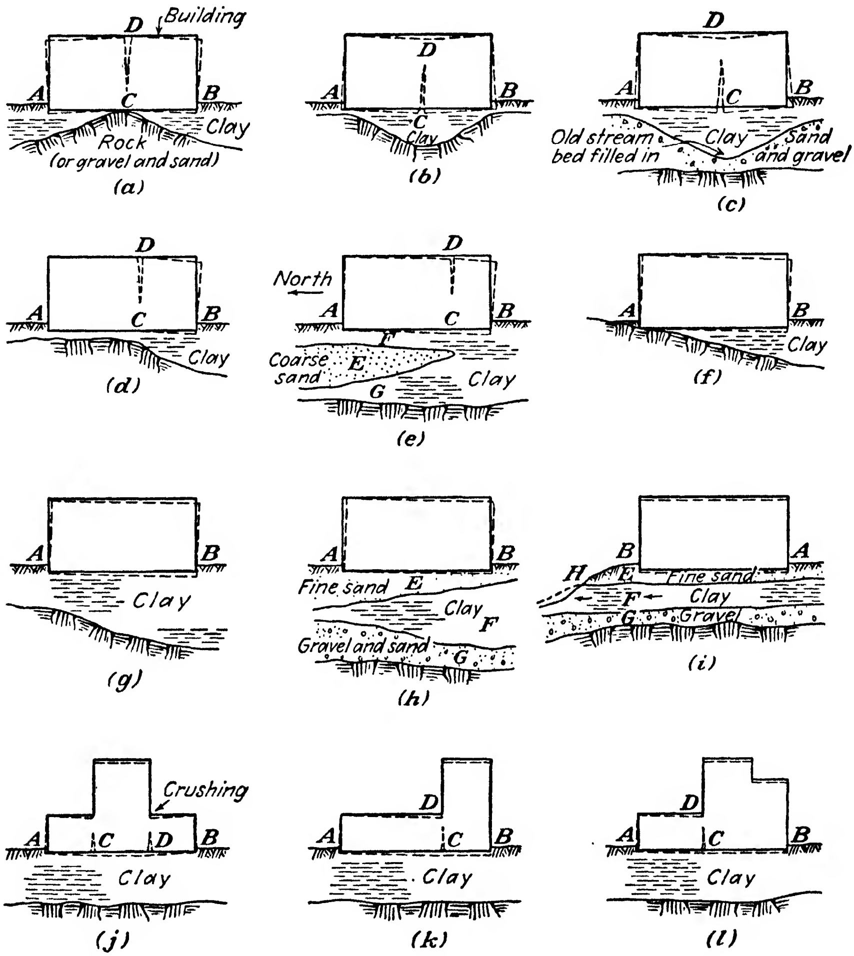


FIG. 4-16. Examples of causes of unequal settlements of portions of a structure.

materials. It is possible to have gravel or sand lenses within or between softer strata in glaciated regions and in places where floods may cause local scouring and deposition of materials. Such conditions are also conducive to unequal settlements. Some such conditions are pictured in Fig. 4-16, with the possible character of the settlement and the effect upon the structure shown to exaggerated scale.

The following comments apply to the various sketches in Fig. 4-16:

(*a*) This pictures a typical case in which settlements can be expected to vary somewhat as the depth of the clay beneath the substructure. The building should be founded upon rock throughout, or a complete contraction or stress-relieving joint should be provided at *CD* if cracking at *D* is to be avoided.

(*b, c*) Here the compressible soil will cause sagging near the center *C*, with possible cracking there. A joint at *CD* is not likely to be very effective. A small structure might be made strong enough to span across such a weak area, but it is usually impracticable to make a large structure capable of doing this. The substructure should be supported upon the firm material.

(*d*) The settlement of the portion *CB* of this structure will make it desirable to have a joint at *CD* or to support the section *CB* on the rock.

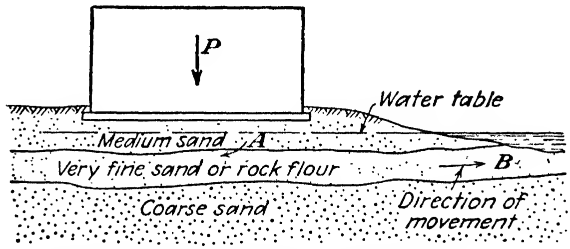
(*e*) Here it is difficult to tell just what the action of the tapered layer of coarse sand *E* will be. It seems reasonable to expect that it will serve somewhat as a soil mat to spread the loads from *F* over a larger area of the underlying clay *G* than will be the case under end *B*. It is therefore likely that the end *CB* will settle so as to cause a crack near *D*. Since the rock is too deep to be reached easily and since the sand stratum does not extend under the entire structure, it seems that there are only three sensible courses to be considered: (1) move the structure farther to the north where the sand stratum *E* is thick and uniform and where the clay layers *F* and *G* will not cause large and unequal settlements; (2) make the structure wide, low, and light so that none of the settlements will be serious; and (3) cut the structure into two or more sections by means of vertical joints so that deformations can occur at these predetermined points. The joints should, however, be keyed or seated so as to transfer the necessary shearing forces without vertical displacement at these joints.

(*f, g, h*) These are conditions that are likely to cause tilting of a structure. In (*f*), the substructure might be carried to rock; in (*g*), piles might be used well down into the clay in order to minimize or eliminate the differential thickness of the compressible layer below the limits of the substructure; in (*h*), piles might be used through *E* and *F* into *G*, or the building might be moved off the tapered clay layer entirely.

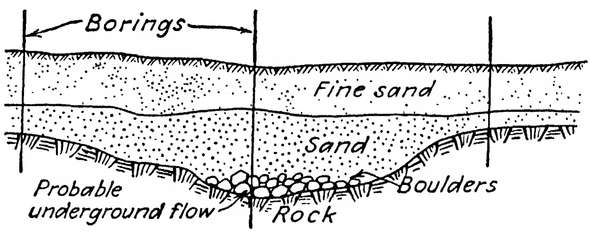
(*i*) In this case, the chief danger lies in the possibility that the clay of stratum *F* may move slowly under pressure toward *H* so that the structure would tilt toward end *B*. The best remedy here may be to set the building farther back from the face of the hillside or to make the structure so that the superimposed load on the soil is small.

(*j, k, l*) These illustrate cases where the unit pressures under the towers are much larger than those under the remainder. This is inadvisable when the buildings are founded upon plastic soils. The shape of the structure should be changed to eliminate this differential, or the substructure should extend to suitable firm materials.

The results of unequal settlement caused by founding one part of a structure upon firm ground whereas the remainder is supported upon newly placed fill can be very serious. A large steel oil tank was built on sloping terrain near the water line alongside the approach to a pier. It was an addition to a tank farm that had been built previously and was near the pump house. Approximately one-third of the tank was supported upon a wedge-shaped layer of new fill. The tank failed suddenly when filled with water for testing purposes. The accident caused a disastrous fire and loss of



(a)-CONDITIONS THAT MAY CAUSE QUICKSAND



(b)-CONDITIONS INDICATING PROBABLE SUBTERRANEAN STREAM

FIG. 4-17. Dangers may exist even in granular soil.

human life. It seems that the fill contained "junk" that was poorly compacted, and so settled rapidly. Apparently, the side plates near the bottom were distorted seriously, and a long strip failed suddenly. Although steel is ductile, there is likely to be a limit to the strains and stresses that it can withstand safely.

In Fig. 4-17(a) is pictured a situation that may cause trouble. The stratum of very fine saturated sand may become quick when loaded by the weight of a structure, causing the sand to move out from A and boil up at B. If the sand under the building were confined by a sort of steel sheet-pile cofferdam whose top was tied together so that the sides could not spread and the sand escape, the sand would support considerable load. Even confined water will do so. Such a remedy, however, may not be so reliable as piles would be if they were driven into the coarse sand.

Another feature to be guarded against is underground streams. When deep excavations are to be made, the flow in a case such as that shown in Fig. 4-17(*b*) may cause considerable trouble and expense during construction. Clean gravel and boulders in a subterranean ravine generally indicate the presence of intermittently or continuously flowing water.

A valley containing a sluggish meandering stream not only may contain soft materials but may hide old stream beds below the surface in different locations so that the soil will not be uniform. Deltas at the mouths of streams and ocean beaches are also sites of probable irregularities and of possible future scouring or deposition of soils.

Obviously these principles of harm because of settlement apply not only to buildings but also to bridges and other structures. The planning of structures to meet many of these conditions is discussed more fully in subsequent chapters.

4-8. Allowable bearing pressures. In order to prepare the contract drawings for the foundations of a structure, someone must determine what is to be used as the allowable unit bearing pressure for purposes of design. This is a very important matter. Local building codes may set values that are not to be exceeded. They are helpful guides because they are usually based upon experience with the soils in that vicinity. They do not, however, protect one against dangerous situations.

The following is quoted from the Foundation Code of the City of New York, revised 1948, except for the addition of the table numbers, which has been done for convenient reference:[1]

§ C26-377.0 Presumptive bearing capacities of soils. *a*. Satisfactory bearing materials shall be ledge rock in its natural bed, natural deposits of gravel, sand, compact inorganic silt, or clay or any combination of these materials. These bearing materials shall not contain an appreciable amount of organic matter or other unsatisfactory material, nor shall they be underlaid by layers of such unsatisfactory materials of appreciable thickness.

b. Fill material, mud, muck, peat, organic silt, loose inorganic silt, and soft clay shall be considered as unsatisfactory bearing materials and shall be treated as having no presumptive bearing value.

c. The maximum allowable presumptive bearing values for satisfactory bearing materials shall, except for pile foundations (see section C26-405.0,*c*), in the absence of satisfactory load tests or other evidence, be those established in the following classification:

Hard sound rock is rock such as Fordham gneiss, Ravenswood gneiss and trap rock, in sound condition, with some cracks allowed.

Medium hard rock is rock such as Inwood limestone, Manhattan schist and massive serpentine with some cracks allowed and slight weathering along cracks.

[1] Courtesy of Councilman Hugh Quinn, head of the New York City Council's Committee on Buildings, who has been the leader in the revision of the Code.

TABLE 4-2. Classification of Supporting Soils

Class	Material	Maximum allowable presumptive bearing values, tons per ft.^{2}
1	Hard sound rock	60
2	Medium hard rock	40
3	Hardpan overlaying rock	12
4	Compact gravel and boulder-gravel formations; very compact sandy gravel	10
5	Soft rock	8
6	Loose gravel and sandy gravel; compact sand and gravelly sand; very compact sand-inorganic silt soils	6
7	Hard dry consolidated clay	5
8	Loose coarse to medium sand; medium compact fine sand	4
9	Compact sand-clay soils	3
10	Loose fine sand; medium compact sand-inorganic silt soils	2
11	Firm or stiff clay	1.5
12	Loose saturated sand-clay soils; medium soft clay	1

TABLE 4-3. Explanation of Terms

Descriptive term	Blows per foot	Remarks
Compaction related to spoon blows; sand		
Loose	15 or less	These figures approximate for medium sand, 2½-in. spoon, 300-lb. hammer, 18-in. fall. Coarser soil requires more blows, finer material, fewer blows
Compact	16 to 50	
Very compact	50 or more	
Consistency related to spoon blows; mud, clay, etc.		
Very soft	Push to 2	Molded with relatively slight finger pressure
Soft	3 to 10	
Stiff	11 to 30	Molded with substantial finger pressure; might be removed by spading
Hard	30 or more	Not molded by fingers, or with extreme difficulty; might require picking for removal

Soft rock is rock such as shale, decomposed serpentine, decomposed schist or decomposed gneiss, with some disintegration and softening and with considerable cracks allowed.

Hardpan overlaying rock is a natural deposit of a thoroughly cemented mixture of sand and pebbles, or of sand, pebbles and clay, with or without a mixture of boulders and difficult to remove by picking.

TABLE 4-4. Soil Sizes

Descriptive term	Pass sieve No.	Retained sieve No.	Size range
Clay...........	200	Hydrometer	0.006 mm.
Silt	200	analysis	0.006 to 0.074 mm.
Fine sand.......	65	200	0.074 to 0.208 mm.
Medium sand....	28	65	0.208 to 0.589 mm.
Coarse sand.....	8	28	0.589 to 2.362 mm.
Gravel.........	...	8	2.362 mm.
Pebble.........	...	....	2.362 mm. to 2½ in.
Cobble.........	...	....	2½ to 6 in.
Boulder.........	...	....	6 in.

d. When it is shown by borings, or otherwise, that materials of varying bearing values must be used for the support of structures:

1. The bearing value allowable for footings on the stronger material shall be unchanged;

2. The bearing value allowable for footings on the weaker material shall be unchanged, provided the weaker material is not more than two classes below that of the stronger material as established in this section, but

3. If the weaker material is ranked more than two classes below that of the stronger material as established in this section, the bearing value allowable for footings on the weaker material shall be reduced by a percentage equal to five times the number of classes it is below the stronger material in ranking.

Another set of data is given in Table 4-5. It varies from the New York Code because it represents a different opinion. In general, it is somewhat more conservative but not necessarily more correct.

TABLE 4-5. Approximate Allowable Bearing Value of Foundation Materials

Material	Bearing capacity, tons per ft.^{2}	
	Approximate depth, 3 ft.	Approximate depth, 6 to 10 ft.
Soft silt and mud..................	0.1–0.2	0.2–0.5
Silt (wet but confined)..............	1–2	1.5–2
Soft clay.........................	1–1.5	1–1.5
Dense firm clay....................	2–2.5	2.5–3
Clay and sand mixed (firm).........	2–3	2.5–3.5
Fine sand (wet but confined).........	2	2–3
Coarse sand......................	3	3–4
Gravel and coarse sand............	4–5	5–6
Cemented gravel and coarse sand....	5–6	6–8
Poor rock.........................	7–10	7–10
Sound bedrock.....................	20–40	20–40

The psychological reactions of people are sometimes peculiar. Let someone write a letter stating that the soil at a certain site is "good for 4 tons per ft.²," and the recipients of this information generally use it as absolutely reliable. They may have many arguments among themselves as to the details of the proposed substructure, but they seldom stop to question the accuracy of this basic starting point for their planning. However, the safe bearing value to be used in the design is the point that really needs the careful study and verification.

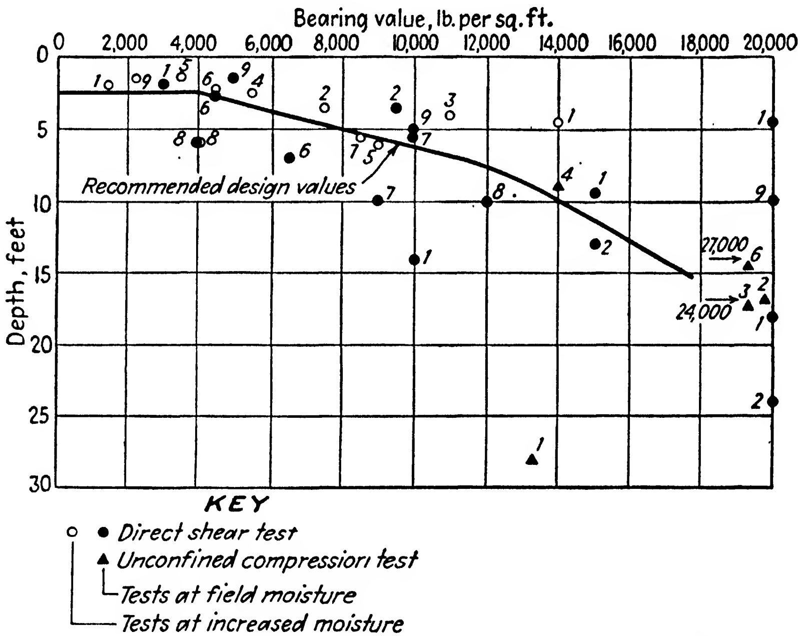


FIG. 4-18. Example of results of laboratory tests to determine the allowable bearing value of a soil. (Courtesy of Dames and Moore.)

Load tests of soils in the field are one source of helpful data upon which to make a decision regarding this vexing problem. Laboratory tests may also be useful.

Figure 4-18 pictures the results of laboratory tests made upon undisturbed soil samples taken at the site of an industrial plant. The results vary, of course; and the plot somewhat resembles a "shotgun" diagram. However, from them the curve shown in the illustration was drawn in order to give data regarding allowable bearing values to be used for design purposes. This curve shows how the bearing value was allowed to increase with the depth of the footings. The tests were helpful in showing that this could be done, and in setting the values. Load tests in the field were not considered necessary since the ground was fairly strong.

The importance of keeping in mind the *scale* of a load test, especially on a plastic cohesive soil, cannot be emphasized too strongly. The magnitude of the test load is negligible as far as the earth mass in general is concerned, and its important effects are localized. Suitable analogies are difficult to find. However, the following may give some notion of the basic idea:

Assume that the reader has a 30-lb. weight fastened to a loop of heavy cord. He hooks one finger into the loop and finds that he can lift and hold the weight safely off the floor with this one finger and that he can carry it around the room. Therefore, he concludes that he can hook a similar weight to each of the four fingers of each hand—not counting the thumbs—and that he can carry all 240 lb. It will not take him long to realize that the strength of his arms, shoulders, back, and legs—his body as a unit—now enters the picture and determines what he can and cannot support.

Similarly, when one makes load tests of the ground, he should not fail to obtain a good scale upon which to compare his test loads to those to be applied by the structure. The tests are helpful in showing the strength or yielding of the soil locally. They have little value in determining the action of deep layers, of long-term consolidation, and of the effects of heavy loads over large areas. However, if the ground cannot support such small loads safely, it will perform even worse under large loads applied perhaps permanently.

The allowable bearing value of a cohesive soil for purposes of design may depend somewhat upon the character of the loading. The dead load is always present. In a warehouse, a large portion of the live load may also be present for considerable periods of time. However, as stated in connection with settlements, ordinary live loads on a bridge, in an office building, on a pavement, or in a factory are temporary. So are wind loads, even though they last for a whole day. These things should be considered when one estimates the permissible bearing value to use in design. For example, clay may resist safely a short-time load of large magnitude, whereas it would be unable to do so if the load were applied continuously. Therefore, when an engineer determines the safe bearing value of a soil, he should not only consider the values specified in codes and indicated by tests but should use good judgment in considering his special structure and the service expected of it, being conservative when circumstances show such a course to be wise.

For example, an industrial plant was built on a fairly dense clay. There were many crane columns having large crane loads but relatively small dead loads. Alongside part of one row was a large heavy elevated bin supported upon separate columns. For design purposes, a bearing of 8 k.s.f. was allowed under the footings of the crane columns but only 6 k.s.f. under those supporting the bin. The smaller bearing value for the soil under the bin was used because of the fact that heavy loads would be applied for long periods, whereas the crane loads would be temporary.

The desirability of thinking in terms of the total load when studying a foundation on plastic soil is well illustrated by the following:

An oil company had a steel tank 40 ft. in diameter and 40 ft. high that was built directly on the ground near tidewater. The soil was plastic clay which had supported this tank safely for many years. The company decided to increase its storage capacity at this port by removing the small tank and erecting in its stead a tank 140 ft. in diameter and 40 ft. high. Those responsible for the design reasoned that the unit pressure under the bottom of the new construction would be the same as under the old tank and that the new one would be safe because the old one had been so.

The construction of the sides of the new tank was performed by erecting the annular rings one at a time. To avoid the use of costly scaffolding, the staging was designed so that it would float in the tank when the latter was partly filled with water. Thus, as each lift of plates was erected and attached, the water level in the tank was raised accordingly, the platforms rose automatically, and the construction proceeded step by step. One day, when the wall of the tank was approximately 75 per cent completed, a little subsidence was noticed. That night the tank settled nearly 16 ft. Apparently, this total load was approximately $0.75 \times 140^2/40^2 = 9.2$ times that of the small tank when filled. Even though this was spread over a larger surface area, the effect upon the underlying soil mass was obviously much more severe than was the original load.

This case is cited to remind one that, when building upon plastic soils, he should try to visualize the structure and the soil as an entity. In general, he should remember that it may be advisable to reduce the theoretically allowable unit bearing pressure as the magnitude of the loaded area increases. Several times, case histories of oil tanks have been used to illustrate particular points. This is not done because oil tanks are dangerous structures but because it is easy to visualize the magnitude of their loads, the uniformity of their distribution, and the actions of the structures.

There may be uncertainty about the load to be permitted upon a large area when the deeper strata of soils are plastic or otherwise weak. About the best that can be done is to make laboratory tests of undisturbed samples of these soils and estimate their strength and probable consolidation under various pressures. Then an allowable superimposed pressure may be selected that is to be permitted upon any particular stratum, and the general scale of the expected settlement can be estimated. If the latter is not harmful, one can estimate backward from the pressure on the lower soil to determine what he will allow at the surface so that, when the latter is distributed downward, the critical material will be loaded approximately as he wishes.

Throughout all the considerations preceding a determination of the allowable bearing value for a particular site should be the question of economy as well as that of safety. Nevertheless, bear this in mind: the cost of the foundation is only one part of the total cost of the project, and a percentage variation in this cost does not have the same relative effect upon the total expenditure, but the safety of the entire structure will depend upon that foundation.

PROBLEMS

4-1. Figure 4-19 shows pressure–consolidation diagrams obtained from laboratory tests of clay soils. Convert these into curves giving the relation between pressure and settlement per foot of soil. Use eq. (4-3).

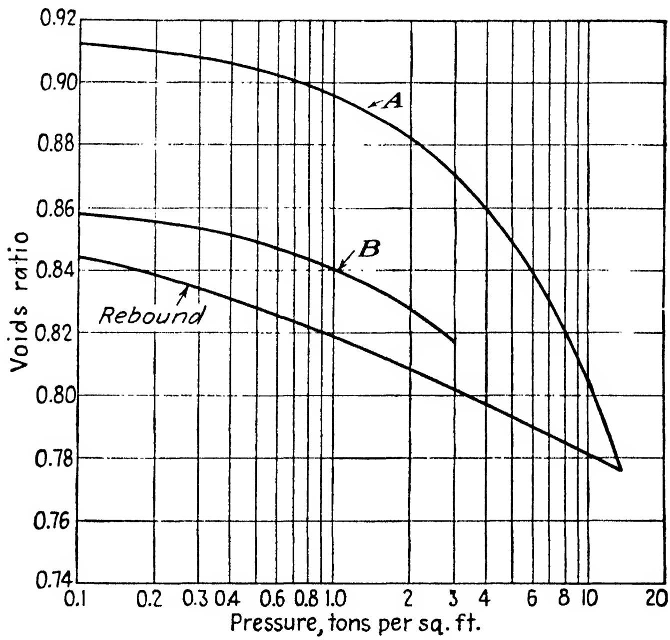
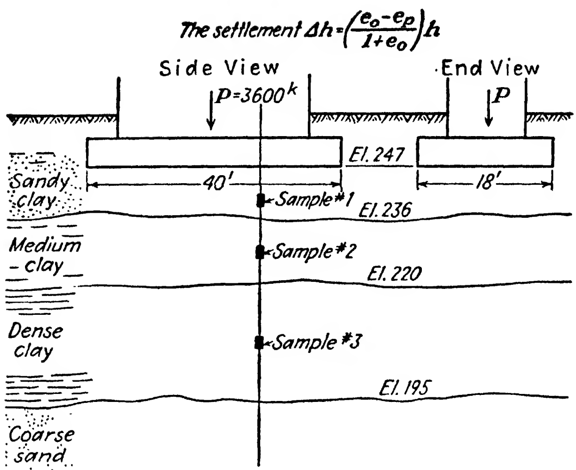


FIG. 4-19. Curves showing character of pressure-voids ratios relationship for two soils.

4-2. Having curve *A* of Fig. 4-19 and the results of Prob. 4-1, estimate the settlement of a building 50 × 100 ft. in plan on a mat foundation resting upon clay 100 ft. deep. The average pressure under the mat is 2,000 p.s.f. Use the 2:1 assumption for distribution of pressures.

4-3. Assume a building 100 × 150 ft. in plan having an average bearing pressure of 1,500 p.s.f. By use of the Newmark circles, estimate the pressure under its center and under one corner at depths of 20 and 50 ft. below the foundation.

4-4. The footing of a bridge pier is to be 25 ft. wide and 50 ft. long. The average bearing pressure is 2.5 tons per ft.^{2} Under the footing is 20 ft. of good sand, then a layer of clay 12 ft. deep, and finally deep sand. If the clay has the characteristics shown in Fig. 4-19, curve *B*, estimate the settlement that may be caused by the stratum of clay. Use the 2:1 method.



Sample No.	Pressure, lb. per sq. ft.		
	0	2000	4000
1	0.929	0.894	0.810
2	0.872	0.849	0.842
3	0.850	0.837	0.828

FIG. 4–20. A problem for estimating settlement of a large pier.

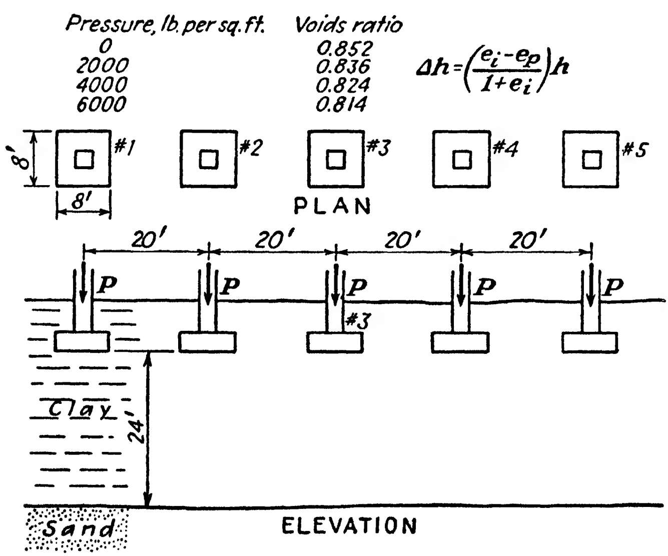


FIG. 4-21. A problem on settlement under small footings.

4-5. Estimate the settlement of the structure shown in Fig. 4-20, assuming 2:1 distribution and the data shown on the drawing.

4-6. Estimate the settlement of footing No. 3 of a series of column footings shown in Fig. 4-21. Each column has a load of 300 kips, including the footing. Consider the clay but not the sand. The clay is uniform in quality, and the voids ratios are as shown on the drawing.

5

SPREAD FOOTINGS

5-1. Introduction. The purpose of this chapter is to show various constructions that may be used for footings supported directly upon soil. Methods are given for proportioning and analyzing these foundations. The reader may think that the methods suggested here and in other chapters are empirical and that the results are approximations. That is true, but they are believed to yield safe results, to be easily understood, and to be simple in application. They are probably just as reliable as are the broad assumptions upon which foundation designs are generally based. Fussy computations may produce excellent psychological results upon him who makes or observes them, yet they should not be expected to correct the approximations in the basic data from which they started.

It is well for an engineer to visualize his structures as though they were made of material that deforms greatly under load action. He should imagine how a contemplated part may fail, then make sure that it cannot fail that way, or any other way. If he foresees all these possibilities and if he takes precautions to prevent harmful results, his structure will probably be safe. In general, if he provides a practicable way in which a structure can act to withstand the loads safely, it will act in that manner rather than fail.

There are many important matters affecting the choice of any one type of foundation for a specific structure. There may be conditions that require the use of one type under part of a structure, whereas another type is preferable under the remainder. It is for the engineer to select that which serves him best and most economically. Such questions cannot be answered specifically for any and all problems, but many suggestions can be given in the hope that they will be helpful to the reader who must make a decision.

Spread footings are used for the purpose of distributing concentrated loads over sufficient soil to enable the latter to support the loads safely. The principles used in designing them are applicable to many other forms of substructure.

5-2. Data for the analysis of reinforced concrete. Many of the computations involved in proportioning substructures require the analysis of tentatively selected members made of reinforced concrete. To save space, therefore, and to have all calculations made upon the same basis, the symbols given in Table 1 in the Appendix are used throughout this book. The permissible unit stresses in materials are assumed to be as shown in Table 2 of the Appendix. These are based upon concrete having an ultimate 28-day compressive strength of 3,000 p.s.i. This will give the reader a uniform datum from which to compare the results of various calculations. In practice, he may use whatever materials and allowable unit stresses are desirable for each particular case, but the principles and methods illustrated remain the same. The approximate formulas that will be used frequently are listed in Table 1 of the Appendix.

Some persons may wish to use larger allowable unit stresses in steel and in concrete than those given here. However, the author believes that stiffness is one of the attributes of good foundations. High unit stresses in reinforcement are accompanied by correspondingly severe cracking of the concrete if the steel is fully stressed, and their use generally effects very tiny total savings whereas they may greatly reduce the reserve strength of the structure. The steel is generally critical in such short thick heavily loaded members as foundations; diagonal tension, shear, and bond may be critical also. Parts of concrete that are stressed highly in compression as beams are generally too thin for practical foundation purposes. One cannot afford to take chances with members that compose a substructure because the results of failure are so serious and repairs are so costly.

5-3. Pedestals. In many instances columns are supported upon concrete pedestals that transmit the column loads to the footings, or to other parts of the substructure. Two such cases are pictured in Figs. 5-1(*a*) and (*b*). Sometimes the limitations of space restrict the area of the top of a pedestal; when practicable, however, it is desirable to have this top equal to at least 1½ or 2 times the area loaded directly by the column if the sides of the pedestal are vertical and the load is large.

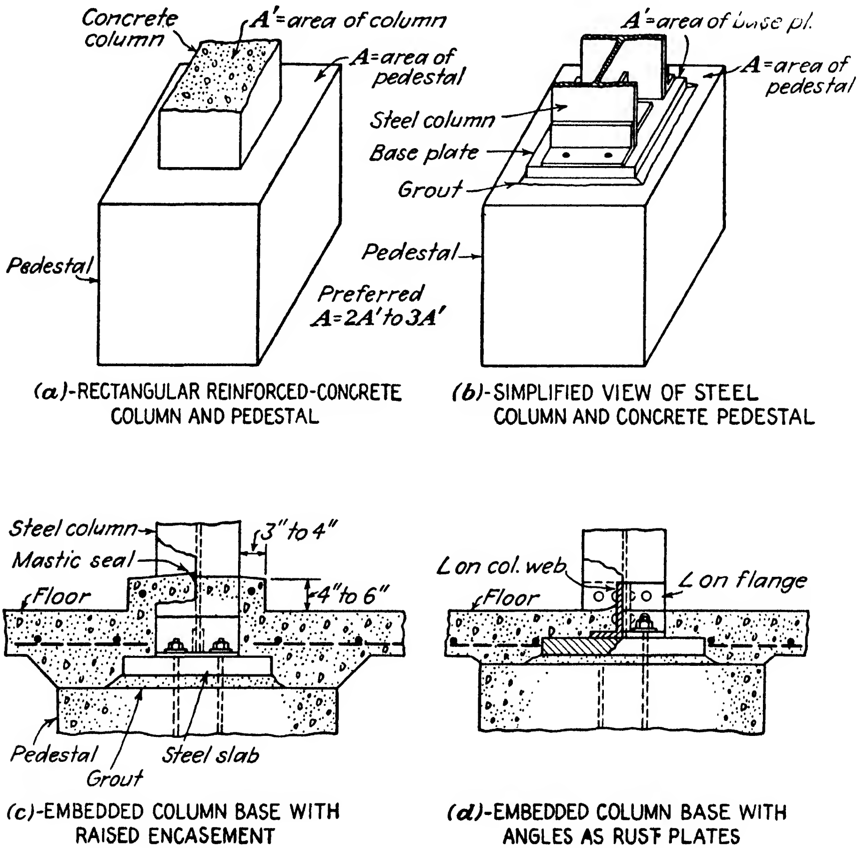
In practice, the following points should be considered when a pedestal is dimensioned:

1. It is desirable to have the compressive stress in a pedestal considerably less than $0.25f'_c$ unless it is beneath a reinforced-concrete column and is itself reinforced as a column.

2. It is almost essential to place the bases of steel columns above the ground and above the water line. Because of possible rusting caused by washing the floor, it may be desirable to place such bases 2 to 6 in. above the ground floor of an industrial plant. The top of the pedestal should then be proportioned in both these cases so as to provide such an increase of area as horizontal clearances and satisfactory appearance make desirable. A

suggested treatment for the bottom of a steel column that must be embedded below the floor line is given in Fig. 5-1(*c*). In Sketch (*d*), the long legs of the base angles are used as "rust plates" to protect the main shaft somewhat against corrosion.

3. The cross section of a pedestal should be ample to spread the load over a suitable area of the footing or slab that is under it.



(*a*)-RECTANGULAR REINFORCED-CONCRETE COLUMN AND PEDESTAL

(*b*)-SIMPLIFIED VIEW OF STEEL COLUMN AND CONCRETE PEDESTAL

(*c*)-EMBEDDED COLUMN BASE WITH RAISED ENCASEMENT

(*d*)-EMBEDDED COLUMN BASE WITH ANGLES AS RUST PLATES

FIG. 5-1. Details at tops of pedestals under columns.

4. A pedestal is supposed to be an enlarged extension of the column. It should therefore be strong and rigid. The depth should be so great relatively that there is no likelihood of cracking of the pedestal because of bending or shear. In Fig. 5-2(*a*), the offset and the depth of the pedestal should be such that angle *ABC* equals 30° or less; the absolute maximum value of this angle should be 45°, which may be satisfactory for small footings.

5. When the column loads are light and the unit stresses in the pedestal are low, the area of a pedestal may be relatively small compared to that of

the column; when the column loads are heavy, it should be more conservative.

6. Although the formwork is more costly, pedestals may be battered as indicated in Fig. 5-2(*b*). With slopes on two opposite sides only, it is simple. If all four sides are to be sloped, one should endeavor to minimize

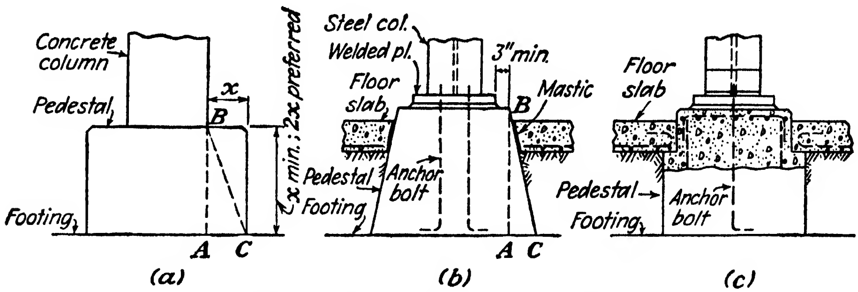


FIG. 5-2. Some types of concrete pedestal.

the variety of sizes of pedestals so that he may use the same forms repeatedly and thus reduce the unit cost.

7. In many cases it may be desirable to seat the floor slabs and beams, too, on the pedestals, as pictured in Fig. 5-2(*c*); in others, where the floor is on filled ground or on plastic soils, it may be better to isolate the slab and the pedestal, as shown in Sketch (*b*).

There is a tendency to crack the upper corners of a pedestal as pictured in Fig. 5-3(*a*), especially when the column loads are large. The dowels shown should equal the area of the longitudinal reinforcement in the column and should be developed by bond. One can realize easily that a hook at the

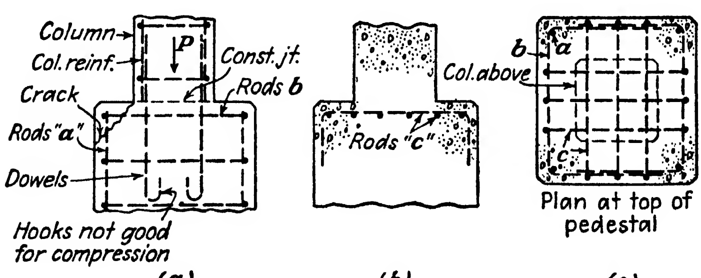


FIG. 5-3. Reinforcement at top of a pedestal.

bottom of such a compression dowel does no special good but may aggravate buckling of the dowel above it.

Assume that a heavy column is supported upon a pedestal and that special means are to be employed to prevent harmful cracking at the abrupt change in section pictured in Fig. 5-3(*a*). If the vertical rods *a* are used near the surface of the pedestal, they will not accomplish much good. Hoops like b may tie in the corners of the pedestal but the straight parts of the rod along the sides will bend outward too easily to offer much resistance to sideward forces. A mat of rods like c in Fig. 5-3(b) will hold these top corners more strongly — at least to prevent appreciable displacement if cracks occur. It is even more desirable to use a combination of reinforcement such as that pictured in Sketch (c) where rods b hold the corners and provide circumferential reinforcement, rods c tie in the central portions, and rods a act as supporting columns for holding the others during the concreting.

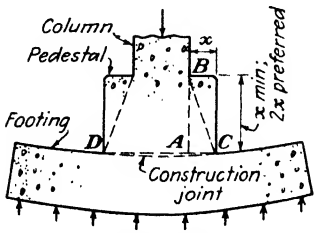


FIG. 5-4. Exaggerated picture of the action of a spread footing.

Since the centralized load on top of a spread footing and the distributed pressure below it cause a footing to try to bend to a saucerlike shape, the top of the footing at DC, Fig. 5-4, shortens and curves slightly, tending to cause a concentration of pressure at the edges D and C. If these vertical pressures were to bend the pedestal, the base DC would lengthen. There is thus a tendency to break the junction DC. On this account, a roughened construction joint is desirable at DC. Such mechanical bond, together with the frictional resistance caused by the vertical pressure, will generally prevent shearing of the junction. Furthermore, the unit compression in the top of the footing at DC, caused by bending, will probably be larger than the theoretical tension in the bottom of the pedestal, and the former will actually prevent tension in the bottom of the pedestal itself. The large edge pressures at C and D, however, make it desirable to have the pedestal sufficiently deep to eliminate weakness in shear.

5-4. Pedestal footings. The most simple individual footing is a solid rectangular unreinforced concrete block like that shown in Fig. 5-5(a). It may be used when the load is light and when a small area of soil will support it safely. In this case, the reacting pressure causes some shear and bending in the footing, but the resultant unit stresses should be small. An exaggerated picture of such a footing tested to failure is shown in Sketch (b).

The pressure under the projecting part $EGBF$ of Fig. 5-5(a) will tend to cause tension near the bottom of EF, and a combination of compression and shear from FB up to the column. The exact magnitude of the shear along EF is indeterminate. Nevertheless, some procedure for analysis of such a footing should be set up in order to have some scale by which to compare and study various footings. The following are suggested as general guides, referring to Fig. 5-5:

1. Limit the angle FEB to 45°, with the vertex E taken at the edge of a concrete column, as in Sketch (a) or, for a steel column, at the base of a

concentric rectangle whose sides are the depth of the column plus $2t$ and the width of the flanges plus $2t$, as pictured in Sketch (c). This seems to be a desirable limitation because of the relative flexibility of the steel billet. Some persons may wish to use unreinforced pedestal footings that are relatively wider than these limits permit. It seems to be unwise to do so, however.

2. Compute the shear that might exist if the pressure on the area $ADCB$ minus $KLMN$ of Sketch (d) were resisted by the total depth EF around the perimeter $KLMN$—what may be called *punching* shear. Limit this

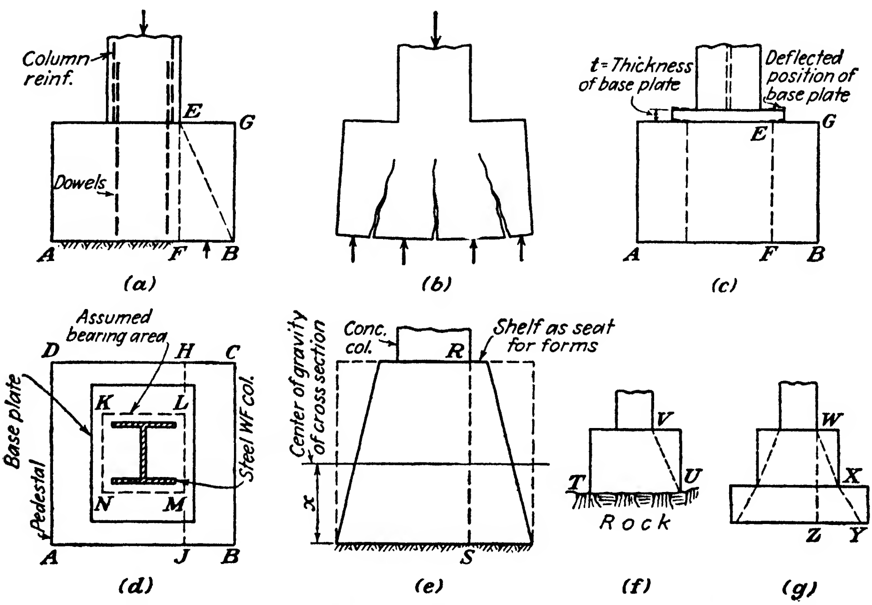


FIG. 5-5. Assumptions used in analysis of plain concrete pedestal footings.

computed shearing stress to $0.05f'_c$. Since all these calculations are based upon assumptions, the full depth of the footing will be used because further refinement is not justified.

3. Compute the bending stress at a section like HJ, Sketch (d), by considering the pressure under $HCBJ$ to act about HJ as an axis, then divide this bending moment by the section modulus of the entire section at HJ. However, limit the computed maximum tensile stress to $0.03f'_c$.

4. If the footing is trapezoidal—a frustum of a pyramid—as pictured in Sketch (e), analyze it for the same punching shear and bending moment as would exist if it were a rectangular vertical-sided footing enclosing the actual one, as shown by the dotted lines. When computing the tensile stress at S, calculate the moment of inertia and the section modulus of the trapezoidal section through RS, using the lever arm x from the center of gravity of the area to point S.

5. A pedestal footing may also be used under a heavy column load if the footing rests directly upon bedrock. If the bond to the rock is trustworthy, the bottom of the footing, TU in Fig. 5-5(f), cannot elongate; therefore, only the punching shear need be computed. However, a little dust or mud on top of the rock when the concrete is poured may destroy most of the bond. Therefore, the footing should be analyzed as though it were on soil, and the limiting 1:1 slope from V to U should still be retained if bearing on the entire bottom of the footing is to be relied upon.

6. If the footing is made as a stepped one, as pictured in Sketch (g), the slopes of lines WX and XY should not exceed 45° with the vertical. The footing should be poured monolithically if it is to be analyzed as an integral stepped rectangular member.

There are many instances where footings are to be designed with the certainty that they will bear on bedrock. There are others where irregular rock is present so that some of the footings of a large structure will rest upon it whereas others will not, unless piers are placed down to it purposely. This last procedure, or the use of piles, may be advisable if the soil overlying the rock is highly compressible and the structure cannot withstand differential settlement without harm.

Does a column of reinforced concrete supported on rock need a footing? Generally, the reinforcement in such a column is used to resist considerable stress. If the rods are merely stopped where the column reaches the rock, they have no reliable way of transmitting their stress to anything except the concrete near the bottom, thus probably overloading it. Furthermore, the rods are generally detailed and fabricated in advance on the assumption that they will start at a given elevation, whereas the rock may not have been excavated at the time and it cannot be blasted out that accurately. Again, such a column is inadequately restrained at its bottom. Therefore it is desirable to use at least a pedestal to receive the column, to contain dowels to transfer the steel stresses into an adequate amount of concrete, to provide a reasonable restraint at the base, to serve as a starting point for the column forms at the desired elevation, and to spread the load sufficiently in case the rock is not of good quality.

When a steel column is supported upon rock, at least a fair-sized concrete pedestal should be used to receive the pressure from the column base, to contain the anchor bolts, to distribute the load a little, and to enable the bases of all columns to be at the right elevation regardless of the irregularities of the rock.

To be considered also is the desired resistance of the base to overturning. In cases where effective restraint is desired, a footing may be needed for this purpose regardless of the ability of the rock to resist bearing.

Example. Is the pedestal footing shown in Fig. 5-6 satisfactory for a concrete column that carries a load of 54 kips?

$$p = \frac{54}{3 \times 3} = 6 \text{ k.s.f.}$$
$$\text{Shear } V = (9 - 1)6 = 48 \text{ kips}$$
$$v_p = \frac{48{,}000}{4 \times 12 \times 18} = 56 \text{ p.s.i. (150 allowed)}$$
$$M = 6 \times 3 \times 0.5 = 9 \text{ ft.-kips}$$

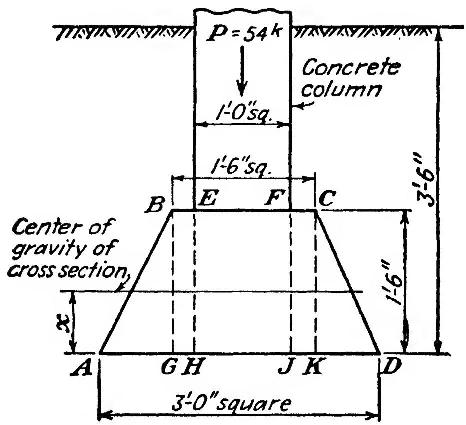


FIG. 5-6. A pedestal footing of plain concrete.

Center of gravity (C.G.) of $ABCD$:

$$x = \left(18 \times 18 \times 9 + 2 \times 9 \times \frac{18}{2} \times 6\right) \div \left(18 \times 18 + 2 \times 9 \times \frac{18}{2}\right) = 8 \text{ in.}$$

$$I_{\text{C.G.}} = \frac{18 \times 18^3}{12} + 18 \times 18 \times 1^2 + \frac{2 \times 9 \times 18^3}{36} + \frac{2 \times 9 \times 18 \times 2^2}{2} = 12{,}650 \text{ in.}^4$$

$$S = \frac{12{,}650}{8} = 1{,}580 \text{ in.}^3$$

$$f_t = \frac{M}{S} = \frac{9{,}000 \times 12}{1{,}580} = 68 \text{ p.s.i. (90 allowed)}$$

The footing appears to be satisfactory from the standpoint of shear and bending.

5-5. Some basic assumptions regarding isolated reinforced-concrete footings. There are three general types of isolated reinforced-concrete footings used under columns and other structural parts that transmit concentrated loads to their foundations: a shallow slab pictured in Fig. 5-7; a sloped footing, in Fig. 5-13; and a stepped footing, in Fig. 5-14.

The first two are used mostly under reinforced-concrete columns, whereas the last may be used with steel or concrete columns. The principal reason for this is the fact that steel columns generally terminate above the floor or ground so that a pedestal is needed to transmit the load from the steel base to the top of the spread footing inasmuch as the latter is usually placed sufficiently deep to be below the frost line and to secure proper embedment. In practice, many combinations of parts are used as substructures.

The word *footing* is used to denote the slab that spreads the load over the soil, and also to mean the local substructure as an entity, including the slab and pedestal. In fact, it is used also in the first sense to refer to the slab under a wall; in the second sense, to denote a combined foundation for two

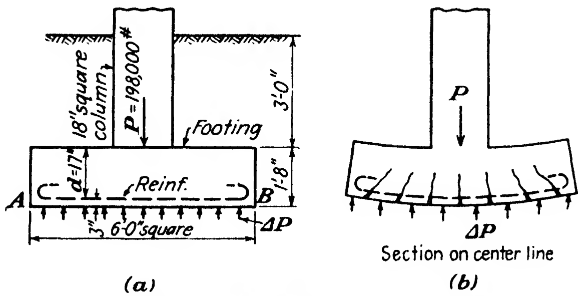


FIG. 5-7. A simple, reinforced-concrete, spread footing.

or more columns as well as for a single one. Isolated footings are also called *spread footings* to differentiate them from wall footings and mats.

Although a spread footing is a very important substructure, its design and analysis are indeterminate and largely empirical. Various engineers have their own ways of designing them. The assumptions and procedures used here for the design and analysis of footings are partly the author's suggestions. They are believed to be logical, easy to understand and apply, and sufficiently conservative to yield safe results.

The pressure from a concentrated load is assumed to be distributed uniformly over the soil in contact with the bottom of the footing if the load is applied at the center of gravity of the footing's bearing area.

The unit pressure upon the soil under a footing, for purposes of estimating the superimposed bearing pressure and the settlements, is generally assumed to be the increase of bearing pressure over that initially caused by the soil above the bearing plane. For example, in Fig. 5-7(a), the added pressure at AB is

$$p = \frac{198{,}000}{36} + 1.67 \times 50 = 5{,}600 \text{ p.s.f. (approx)}$$

The second term in the preceding equation represents the increase in pressure caused by the weight of the footing at 50 p.c.f. in excess of the weight of the displaced earth. This additional pressure is generally of little consequence and is omitted in most cases. If the weight of the soil that was (or is) above AB is always included in computing the effective pressure applied at AB, one will realize that some unwise conclusions may result. For example, an embedment of 33 ft. instead of 3 ft. in Fig. 5-7(a) would add a theoretical pressure of approximately 3,000 p.s.f., thus implying that the footing AB could safely carry less load when deeply embedded than when near the surface of the ground. Only in special cases, such as excavations in plastic soils, silts, and quicksands, need this feature of overburden be a cause for worry. It is generally the *increase* of pressure above that originally present that interests the engineer.

Inasmuch as the concrete of the footing AB, Fig. 5-7(a), is placed upon the ground in a plastic state, the effect of its own weight need not be included in the pressures that cause bending moments and shears in the footing itself. Therefore, in this case, the unit upward pressure for use in the analysis of the footing may be assumed to be the concentrated load applied at the top of the footing divided by the area of the bottom of the footing. For Sketch (a), the effective bearing pressure p is

$$p = \frac{198{,}000}{36} = 5{,}500 \text{ p.s.f.}$$

This pressure causes the footing to deflect somewhat as pictured to exaggerated scale in Fig. 5-7(b). Thus the footing acts as some sort of cantilevered member that bends in radial directions about a centrally located column base. The backfill above the footing is not considered to have any effect upon this action.

5-6. Action of a circular spread footing. To visualize the action of an isolated spread footing, assume first a circular concrete column supported upon a circular flat slab, as pictured in Fig. 5-8. Sketch (a) shows a vertical section or narrow slice along one diameter of such a footing. The pressure under the entire footing tries to curl the slab upward so as to form a saucer-like shape that is concave on top, as shown in the qualitative contour plan in Sketch (b). The footing therefore elongates radially near the bottom, whereas it shortens radially near the top. Rod a in Sketch (a) is used to resist the radial tension; if it resists this tension, it must elongate, and the concrete will probably crack somewhat as shown. Furthermore, the radial elongation of the bottom must be accompanied by a circumferential elongation of the same region. Similarly, the top will have both radial and circumferential compression.

Imagine that the footing is reinforced radially by four rods at 45° angular spacing, as shown in Sketch (c). The plan of some possible hair cracks at the bottom is given also. The sectors $CDEF$ and $HJKG$ of the angular ring outside of the column's perimeter tend to break off and rotate upward about CF and GH as pictured in Sketch (*d*). Rod *a* then serves as a tie to prevent failure in this manner. If hoop *b* were used instead of rod *a*, it might also prevent an excessive outward movement of the bottom and thus enable the footing to resist pressure. At the same time, compression acts upon the uncracked concrete in the vicinity of L and M. Hence, there must be a shearing force acting upon each of these uncracked areas, too,

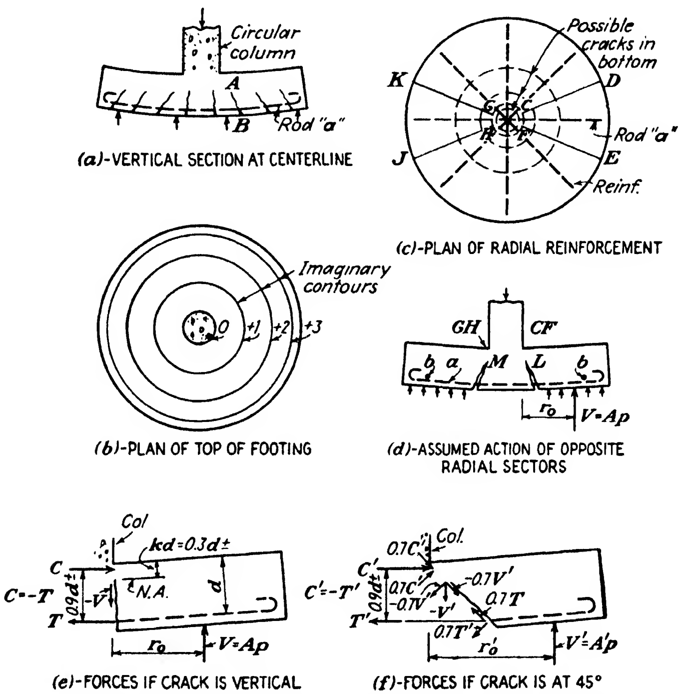


FIG. 5-8. Picturization of assumed action of a circular footing.

because the shear cannot cross the cracks below this area even though they are infinitesimal.

The static forces that are assumed to act upon $CDEF$ are pictured in Sketch (*e*) when the tensile crack below the column's edge is vertical. The radial bending moment at the column is the product of the area of the sector times the unit pressure times the lever arm to the center of gravity of the area ($M = Apr_o$). If the crack is inclined at 45°, the forces that may act upon the remainder of the sector are pictured in Sketch (*f*). The tangential components of C' and T' annul each other; that of V' acts upon the uncracked concrete. The component of V' normal to the sloping plane counteracts part of the corresponding component of C'.

An examination of Figs. 5-8(e) and (f) indicates that the following assumptions may be logical:

1. The critical compression and tension will occur in the plane of the edge of the column when the tensile cracking is vertical. This should be assumed as a critical case for bending moments.

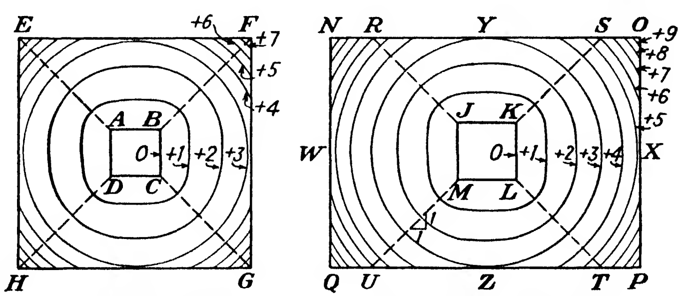
2. The greatest intensity of transverse shear—*punching shear*—will occur in the uncracked concrete at the edge of the column when the tensile cracking is vertical. This is a critical case for punching shear.

3. Any tensile cracking underneath the projected area of the column will probably be vertical because it will be at right angles to the tensile forces, and these are horizontal. Outside of the projected area of the column, however, the cracks may be inclined downward and outward from the neutral axis because of the combination of horizontal tension and vertical shear. As a critical case, the flattest angle of cracking may be assumed to equal 45° with respect to the vertical plane at the face of the column.

4. With 45° cracking, the vertical shear causes a tendency for tensile stresses at the uncracked section to oppose the compressive stresses, thereby decreasing the effective resistance of the section against vertical displacement of the portion beyond this 45° plane. This condition is then a critical case for shear or diagonal tension.

5-7. Square and rectangular isolated spread footings. It is not ordinarily practicable to use circular footings like that in Fig. 5-8(c) under columns because the formwork is more expensive, the radial reinforcement stacks up too much at the center, and bands of parallel rods require the use of various lengths of steel in each band. Therefore, square and rectangular footings are generally used.

A square footing, such as that shown in Fig. 5-7(a), tries to deform somewhat as does a circular one. Compare the contours in Fig. 5-8(b) with



(a)-A SQUARE FOOTING (b)-A RECTANGULAR FOOTING

FIG. 5-9. Imaginary contours to picture curvature of top of square and rectangular footings.

those in Fig. 5-9(a). If the latter footing is not ruptured, the concave upper surface will not have sudden breaks or changes in slope that would be revealed by sharp angles in the contours. It seems logical to assume that the corners of the column *ABCD* will tend to "ride" the top of the footing, and hence they are likely to be points of high pressure. It seems likely also that contours near the column will be rounded squares, whereas those farther away become more nearly circular. The corners of the footing obviously curl upward the most. In an elongated footing like that of Fig. 5-9(*b*), the action is similar to that of a square one except that the ends must deflect more than the sides.

It also seems logical to assume that the centrally loaded square footing in Fig. 5-9(*a*) has equal pressures, shears, and bending moments upon all four symmetrical parts. On the other hand, if lines *KS* and *LT* are drawn from the corners *K* and *L* of Fig. 5-9(*b*) at 45° angles to the sides, it is obvious that more pressure, shear, and bending moment must be caused by part *TLKSOP* than by part *SKJR*. The more elongated a footing becomes, the more unequal will be these two parts and the forces acting upon them.

How, then, can one estimate the shearing and bending stresses in square and rectangular footings? It is not practicable to analyze them exactly as has been suggested for circular footings; nevertheless, a logical, simple, economical, and safe procedure is desirable.

The procedure for, and the psychology underlying, the design and analysis of square and rectangular footings may be outlined briefly as follows:

1. Assume a style or shape of footing that seems to serve the particular purpose required.
2. Determine an adequate bearing area that the footing must have.
3. Assume all thicknesses and other dimensions for the trial footing upon the basis of judgment and precedent.
4. Make the footing so that it cannot fail by bending and shear in the direction *WX* of Fig. 5-9(*b*); make it so that it cannot fail in the direction *YZ* either. If these two paths of resistance are strong enough, the footing cannot fail.
5. Check the design for economy and general suitability.

The report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete of 1940 has specified one method for the computation of the bending moments to be resisted in a footing. It assumes that the projecting portion *KJGH* in Fig. 5-10(*a*) will act as a cantilevered beam about the axis *KJ* at the real or approximated edge of the column or pedestal, and that the side *ELMH* will act similarly about *LM*. Inasmuch as part *KDMH* is thus included twice, the code states that 85 per cent of the computed bending moments and shears are to be used for design purposes. The section *KJ* of the trial footing is then analyzed as a rectangular reinforced-concrete member of width *KJ*. This method is called the *rectangular method*. If the concrete column is circular, an imaginary square $ABCD$ having the same area as the gross cross section of the circular column is substituted for it.

Another method of computing the bending moments and shears to be resisted in a footing, as stated in the 1936 report of the Joint Committee, is based upon the assumption that the projecting portions like $DCGH$ and $ADHE$ in Fig. 5-10(b) cause bending and shear about CD and AD, respectively; and the entire computed bending moment and shear for each portion alone are used in analyzing the trial footing. This method is called the *trapezoidal method*.

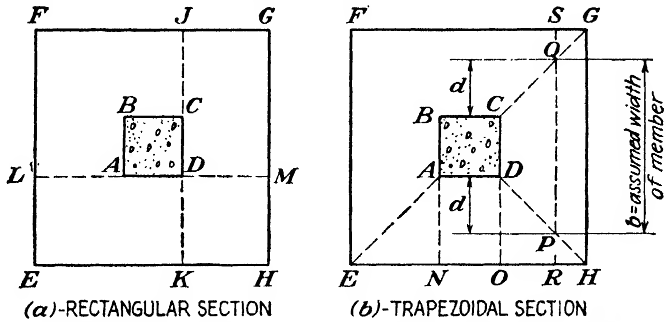


FIG. 5-10. Two methods of computation of bending moments in a reinforced-concrete spread footing.

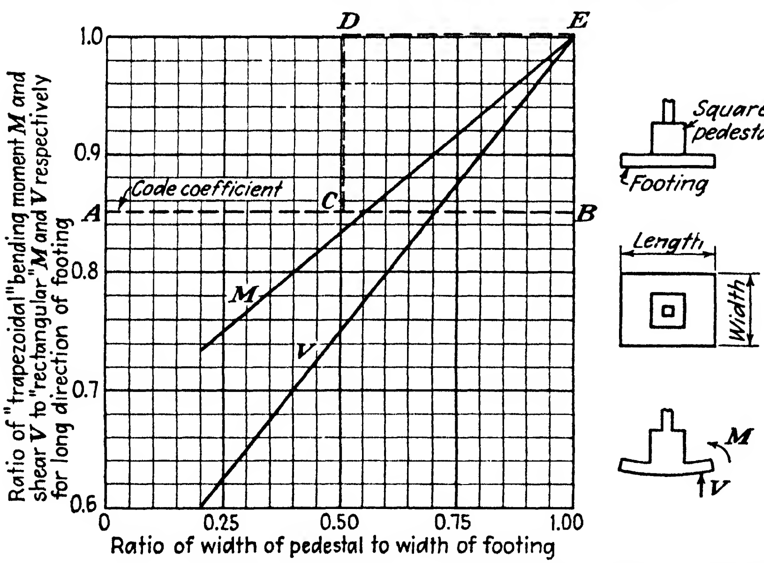


FIG. 5-11. Relative values of assumed bending moments and shears in rectangular footings when computed by "trapezoidal" and "rectangular" methods.

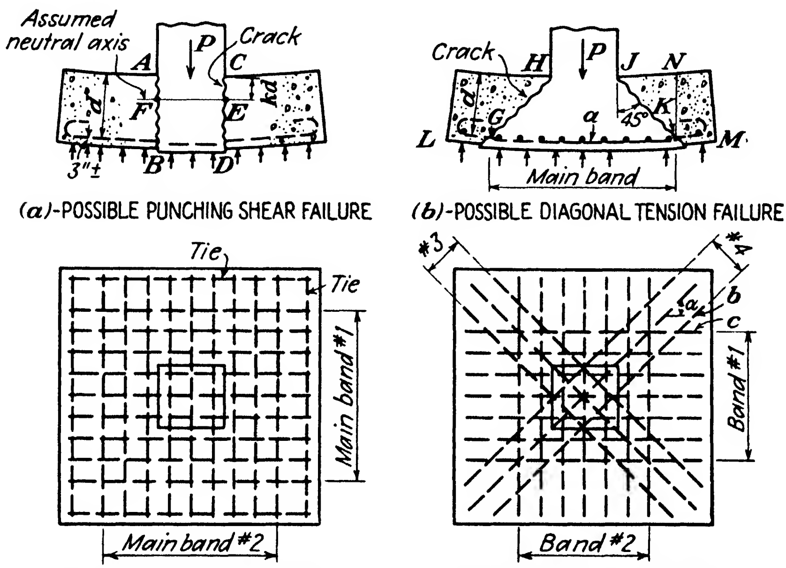
Figure 5-11 has been prepared to show the theoretical relative magnitudes of the bending moments and shears in square and rectangular footings when computed by the rectangular and the trapezoidal methods. Apparently, the relation of the width of the pedestal or step to the width (smaller side dimension) of the footing is the important factor in the results; the relative magnitudes of the bending moments and the shears in the long direction do not vary between the two methods, even though the longer side may be from one to two times the shorter side. From this comparison, it seems desirable to limit the 0.85 coefficient recommended by the code from AB to AC, Fig. 5-11. In other words, the following coefficients are suggested for use with the rectangular method:

1. 0.85 for the bending and shear in the long direction if the width of the square bearing area on the footing is not over one-half the width of the footing.
2. 1.0 for the bending moment and shear in the long direction if the width of the square bearing area on the footing is more than one-half the width of the footing.
3. 0.85 for the bending moment and shear in the short direction when the length of the footing is more than 1.5 times its width.
4. Use values given by the diagram of Fig. 5-11 for each particular case of relative dimensions when the possible economies of material and duplication make it seem desirable to use them. This really is equivalent to the use of the trapezoidal method.

Apparently, the trapezoidal method, or the suggested substitute, seems to be more logical than the rectangular method for square footings that are 10 ft. or larger in plan, whereas the rectangular method may be useful when the footings are elongated and when they are subjected to overturning forces that cause unequal pressures on the bottom.

Seldom does the compression in the concrete due to bending cause critical unit stresses in properly proportioned footings. The 1940 Code assumes that the full width of the footing is effective as a rectangular beam. This is satisfactory for small footings. However, the author prefers to assume that the "effective" width of footing for computing the compressive unit stress and the diagonal tension in large footings be assumed as the width PQ of Fig. 5-10(*b*), a distance equal to the effective depth of the footing out from sides BC and AD, or from the sides AB and CD in the other direction. If the footing is not square, the effective width for analysis of unit stresses may still be taken as $CD + 2d$, and the bending stresses may be computed conservatively upon the basis of this width. This may not be correct, but it generally is a safe procedure.

When footings are placed upon soils of high bearing value, there may be danger of failure because of *punching shear*. This may be especially important when no distributing pedestals are used. The tendency of a column shaft or pedestal to punch through a footing is pictured in Fig. 5-12(*a*), causing failure along such planes as AB and CD. However, since tensile cracking of the concrete probably will have occurred below the neutral axis at E and F because of the elongation of the reinforcement, as explained previously, the uncracked portion kd of the section is all that should be depended upon to resist transverse breaking. The cracks certainly will not be helpful in resisting such action. Furthermore, since the steel rather than the compressive strength of the concrete is critical in most footings, the latter are underreinforced. Therefore, the factor k for finding the assumed location of the neutral axis is relatively small. It is assumed



(a)-POSSIBLE PUNCHING SHEAR FAILURE (b)-POSSIBLE DIAGONAL TENSION FAILURE

(c)-TWO-WAY REINFORCEMENT (d)-FOUR-WAY REINFORCEMENT

FIG. 5-12. Shearing action in, and reinforcement of, a spread footing.

to equal 0.3. For practical reasons and for conservatism, the permissible value of this punching-shear stress should be limited. A correct magnitude for it seems to be unknown, but this transverse shearing stress v_T may safely be at least as high as $0.15f'_c$ or $0.2f'_c$. The latter value is assumed here.

For example, an analysis of the footing shown in Fig. 5-7(a) will give the intensity of punching shear v_T as follows:

$$5{,}500(36 - 1.5^2) = 4 \times 18 \times 0.3 \times 17v_T$$
$$v_T = 510 \text{ p.s.i. } (0.2 \times 3{,}000 = 600 \text{ p.s.i. allowed})$$

Footings are such short heavily loaded members that shearing forces in them attain unusual importance. This concept of punching shear is useful when one checks the assumed depth of a footing. If v_T is excessive, the required amount of tensile reinforcement is probably excessive also.

Another probable weakness of a large footing because of shearing forces has previously been called its resistance to diagonal tension—what may also be called *longitudinal shear*, v_L. Figure 5-12(*b*) pictures such assumed action of a square footing under loads. It has been assumed that diagonal tension may cause cracks at approximately 45° with the vertical, along such planes as GH and JK. Beyond this theoretical frustum of a pyramid, it is possible that such portions as LG and KM may break off, as shown to exaggerated scale in Sketch (*b*). The intensity of the longitudinal shear at K may then be assumed as caused by the pressure on such an area as $PQGH$ of Fig. 5-10(*b*) acting upon a vertical section PQ wide and $jd = 0.9d$ deep if the trapezoidal method is used. If the rectangular method is preferred, the shear acting upon the section will be assumed as the pressure upon area $SRGH$ of Fig. 5-10(*b*), and the width will be RS unless the reader follows the more conservative suggestion that the assumed width be limited to PQ in any case, where PQ is one side of the assumed 45° frustum of a pyramid. It is in this matter of shear particularly that the trapezoidal method of analysis of a footing seems to be a helpful concept.

Inasmuch as it is desirable to make a footing thick enough so that no web reinforcement is required, in order to attain stiffness and simplicity of reinforcement, the permissible magnitude of the longitudinal shear v_L at the edges of the 45° slopes should not exceed $v_L' = 0.03f_c'$. It is, of course, assumed that the tensile reinforcement is always anchored by hooks or other suitable means; otherwise, v_L' should not exceed $0.02f_c'$, and it is probable that the reinforcement will not be fully effective.

If bond between the concrete and the steel cannot develop the required stress in the rods, they will be ineffective, and failure may ensue. Hooking of reinforcement is ordinarily helpful. Some of the new types of deformed bars are apparently better than the old ones in developing bond, and some persons believe that they do not need to be hooked. The cost is so small, however, and the consequences of slippage are likely to be so serious, that one should be cautious about dispensing with hooks in important footings. On the other hand, welded mats of straight rods closely spaced, welded together strongly, and with at least one bar close to each end, produce a reinforcement with a very high degree of mechanical bond so that hooks on these rods are seldom necessary.

Footings that are relatively small compared to their depth may have the tensile reinforcement distributed across the entire width. However, in large footings where the cantilevered projection is 1.5 to 2 or more times the effective depth, it may be preferable to locate the main reinforcement nearer the central portion of the footing. In Fig. 5-12(*b*) it is grouped purposely within a band or critical section that is equal to, or not much wider than, GK—the limits determined by the assumed 45° shear lines from the edges of the column or pedestal, as for the assumed effective width of footing for beam action. One or more tie bars, like a, even though they are not counted as main tensile steel, should be used for reinforcing the corners of the footing and for tying the bottom edges together when the main rods are more than 8 to 10 in. from the edges. This arrangement provides two strong bands as shown in Sketch (c), these bands passing under an area near the column that can be relied upon to provide a trustworthy vertical reaction. The effectiveness of heavy steel near and parallel to the edges of large footings may be questionable because of their tendency to deliver additional loads to the ends of the centrally placed rods in the opposite band if these edge rods really function as intended. Furthermore, the upward curling of a large footing tends to cause the resistances to act through the shortest and stiffest paths, which are or try to be in a radial direction about the center. Hence, it seems best to have the steel relatively near the central portion of large footings.

The four-way arrangement of reinforcement shown in Fig. 5-12(d) may be used, but the four layers of rods require considerable depth for packing up where they cross. Furthermore, the parts of the diagonal bands that are assumed to be effective in assisting the main bands of steel are the components of their areas parallel to the latter. For example, rod b times cos α is relied upon to aid rod c.

Proper spacing and bond stresses of reinforcement are to be secured in any case. In Fig. 5-10(a), the shear for computing the bond stress on the rods crossing JK is the proper coefficient from Fig. 5-11 times the pressure under $KJGH$ when the rectangular method is used. In Fig. 5-10(b), the assumed shear for the bond stress on the rods crossing the direction of CD is the pressure under $HDCG$, using the trapezoidal method.

Some specifications permit the use of a bond unit stress in excess of $0.05f'_c$ when designing two-way footings. For example, the 1940 Report of the Joint Committee allows $0.0375f'_c$ for deformed bars with an increase of 1.5 times this, when special anchorage is used. For 3,000-lb. concrete, this specification would allow 170 p.s.i. This value is used sometimes in the following problems. However, inasmuch as the bond resistance in footings, as in other reinforced-concrete structures, is so important, the author, in his own practice, prefers to design all foundations upon the same basis. Furthermore, it is often difficult to determine just when an allowable unit stress should be based upon the two-way principle and when upon one-way action. The permissible bond unit stress for many numerical problems in the text is, therefore, assumed equal to 150 p.s.i.

The magnitude of j for computing tensile stresses in, and bond stresses along, reinforcement may be assumed equal to 0.88 or 0.9. The latter will be used generally. Because of other uncertainties, no theoretical calculation of j seems justified.

Sloped footings, such as that shown in Fig. 5-13(*a*), are somewhat more economical in their requirements for volume of concrete than are flat-topped ones. They also have greatest depth where the bending moments and shearing forces are the largest, and they are often desirable when no pedestal is to be used. The tangent of the angle δ should not exceed 0.33(1:3 slope) if the use of top forms is to be avoided—even then the con-

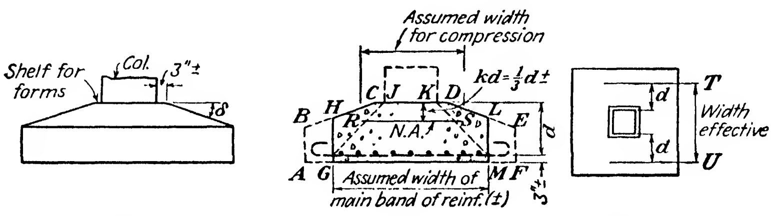


FIG. 5-13. A sloped-top spread footing.

crete should be relatively dry. A maximum top slope of 1:4 is preferable. The assumed width of the main bands of reinforcement may be determined in the same manner as for Fig. 5-12. However, as seen in Fig. 5-13(*b*), the cross section of the assumed cantilevered beam is not rectangular. For estimating roughly the unit compressive stresses, it should be sufficient to assume that the section is a rectangular beam of width *CD* plus one-half the horizontal projections of *CH* and *DL*. The neutral axis may be assumed to be at ⅓d from the top of the footing, this being conservative. If f_c seems to be critical, a more careful analysis may be made.

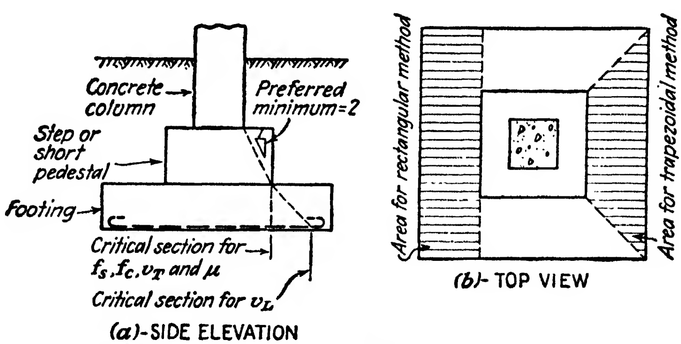


FIG. 5-14. A stepped footing.

For estimating the diagonal tensile stresses in a sloped footing, the pressure under *MF* of Fig. 5-13(*b*) may be assumed to act upon a vertical section that is a rectangle whose width equals *TU* of Sketch (*c*) and whose effective depth is that of the footing at section *LM*.

A stepped footing like that of Fig. 5-14 may be analyzed in a manner similar to that used for the footing in Figs. 5-7, 5-10, and 5-12 except that the edge of the pedestal is used as the axis for computing bending moments, and for reference when estimating shearing forces. The pedestal should be analyzed to see that it is adequate to serve its purpose of spreading the column load over a large area of the footing. It permits the use of a much shallower footing and less steel than would be required for a simple slab footing supporting the same column.

Again it is emphasized that the design of a spread footing involves first the selection of a type that is satisfactory, then the choice of dimensions for it that provide adequate bearing area and other desired features, and finally an analysis of it to determine the reinforcement and to see if the entire unit is safe and satisfactory without being wasteful. A freehand sketch of the trial footing made reasonably to scale on a cross-sectioned piece of notebook paper will generally show an experienced engineer whether or not this tentative substructure is likely to be satisfactory.

One of the strangest misconceptions of the action of a spread footing—or else an illustration of dishonest construction—is the case pictured in Fig. 5-15. A large mill building was built about 1900. The original column foundations were plain-concrete pedestal footings, as shown by *CDFE*. Sometime around 1925 the structure was rebuilt for heavier duty, and the floor and yard were raised to avoid a repetition of past flooding. The new spread footings *AB* were built on top of the old footings.

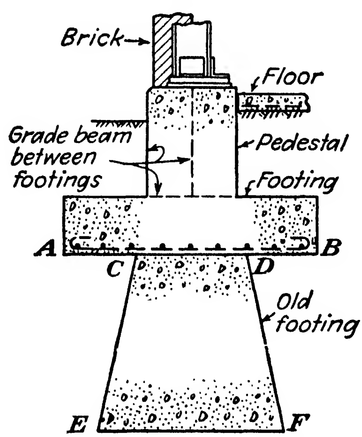


FIG. 5-15. An example of unwise construction.

It is obvious that the soils under *AC* and *DB* are far more compressible than the concrete block under *CD*. Therefore, the column load will be transmitted directly down to the old bearing area at *EF*, and the spread footing will do no good in supporting loads unless the settlement of *EF* becomes so bad that the latter departs for unknown lower regions. Loads are transmitted to the strongest practicable reaction points by the most direct and stiffest path that is available.

The reader can imagine the problems involved when this condition was discovered during the planning of a second remodeling and strengthening of the structure during the Second World War. Some of the foundations had to be surrounded by a wide reinforced-concrete collar under and beyond *AB* and extending down to the level of *EF* in order to have proper bearing area. This made the remodeled footings a sort of enlarged block of concrete of strange composition but of trustworthy safety. The unit cost was very high.

Example 1. Part of the superstructure of a highway viaduct is to be supported upon a concrete tied column that is 2 ft. 2 in. square. The vertical load at the base of the column is 400 kips. The soil profile is shown in Fig. 5-16(*a*). Frost depth is at least 4 ft. Design a footing for this column. Allow 170 p.s.i. for bond.

Lacking more accurate data, the allowable bearing pressure on the soil will be assumed to be 5 kips per ft.^{2} for the sandy clay and 8 kips per ft.^{2}

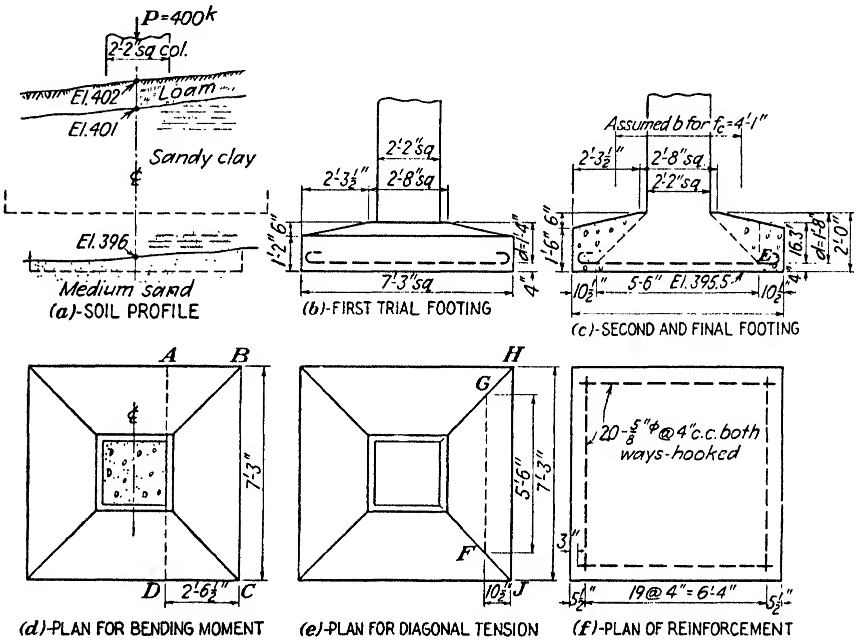


FIG. 5-16. Footing for a viaduct column.

for the medium sand. The approximate required bearing areas are $400/5 = 80$ ft.^{2} for the clay and $400/8 = 50$ ft.^{2} for the sand. The latter material is so much stronger that it seems preferable to use a footing 7 ft. 3 in. square extending to El. (elevation) 395.5 rather than one 9 ft. square at El. 397.5, approximately 4 ft. below ground at its southern edge.

The reinforced-concrete column may be extended into the ground without the use of a separate pedestal if the cover over the reinforcement is sufficient. A sloped-top footing may be suitable and economical. One is therefore assumed and sketched as shown in Fig. 5-16(*b*). For purposes of illustration, use the rectangular method.

In this case, the punching shear is estimated as follows:

$$p_{av} = \frac{400{,}000}{7.25^2} = 7{,}600 \text{ p.s.f.}$$

$$v_T = \frac{400{,}000 - 2.17^2 \times 7{,}600}{kd \times 26 \times 4} = \frac{364{,}000}{0.3 \times 16 \times 26 \times 4} = 730 \text{ p.s.i.}$$

(greater than 600)

Therefore, the footing is probably too thin for other purposes also. A new trial d is

$$d = \frac{364{,}000}{v_T k \times 104} = \frac{364{,}000}{600 \times 0.3 \times 104} = 19.4 \text{ in. (assume } d = 20 \text{ in.)}$$

This new footing is shown in Fig. 5-16(c).

From Fig. 5-16(d), using the rectangular method,

$$V = 0.85 \times 7{,}600 \times 7.25 \times 2.54 = 119{,}000 \text{ lb.}$$

$$M_{AD} = 119{,}000 \times \frac{2.54}{2} = 151{,}000 \text{ ft.-lb.}$$

$$A_s = \frac{M}{f_s jd} = \frac{151{,}000 \times 12}{18{,}000 \times 0.9 \times 20} = 5.6 \text{ in.}^2$$

This footing is relatively so thick that the full width may be used for the placing of the reinforcement. Therefore, use thirteen ¾-in. round rods at 6½ in. c.c. (center to center); A_s = 5.72 in.²; band width = 6 ft. 6 in.

$$u = \frac{V}{(\Sigma o)jd} = \frac{119{,}000}{30.7 \times 0.9 \times 20} = 215 \text{ p.s.i. (greater than 170)}$$

This high bond stress is often troublesome when one analyzes footings. It shows that it is desirable to use small bars close together rather than big rods at large spacing, in order to obtain a relatively large surface area for bond. If ¾-in. rods have to be used because of stock sizes, the number should be increased to provide the necessary surface area. In this case, the required number would be $N = \frac{13 \times 215}{170} = 17$. At 5 in. c.c., the band width = 6 ft. 8 in.

It is better, however, to use twenty ⅝-in. round rods at 4 in. c.c., for which A_s = 6.20 in.², u = 169 p.s.i., and the band width = 6 ft. 4 in.

A check of f_c for an assumed effective width of 4 ft. 1 in. gives

$$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 151{,}000 \times 12}{0.3 \times 0.9 \times 49 \times 20^2} = 680 \text{ p.s.i.}$$

A check of the diagonal tension v_L at E, Fig. 5-16(c), using the trapezoidal method and the area $FGHJ$ of Sketch (e), gives

$$v_L = \frac{V}{bjd} = \frac{\left(\frac{5.5 + 7.25}{2}\right) 0.88 \times 7{,}600}{66 \times 0.9 \times 16.3} = 44 \text{ p.s.i.}$$

Therefore, the footing will be satisfactory if it is dimensioned as shown in Fig. 5-16(*c*) and reinforced as in Sketch (*f*).

Example 2. Figure 5-17 shows an interior steel column that an engineer wishes to use in an industrial plant. Alongside the column is to be part of the foundation of a large machine. It is essential to keep the column's foundation separate from that of the machine. The engineer in charge of the design has ordered the permissible bearing pressure to be limited to

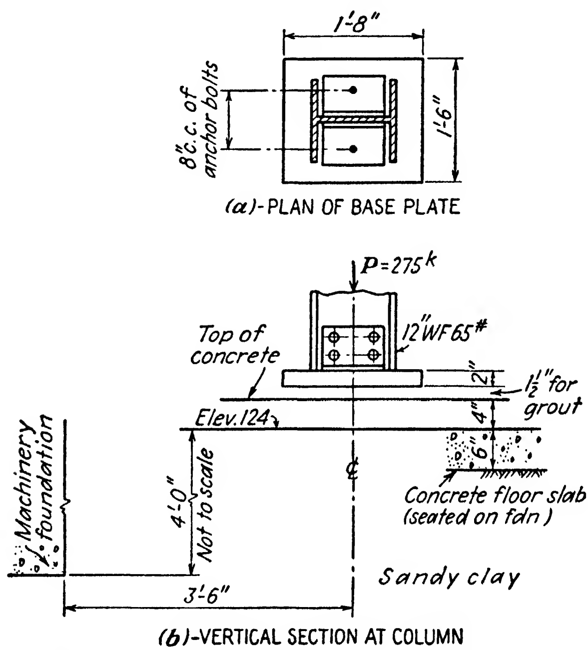


FIG. 5-17. Base of a steel column in an industrial plant.

2 tons per ft.² Design a concrete spread footing to serve this purpose. Let the bond unit stress equal 170 p.s.i.

What type of construction is desirable and practicable? To answer this, consider the following conditions that are necessary or seemingly desirable, referring to Fig. 5-17:

1. This column is in the interior of the building, hence freezing will not control the depth of embedment of the footing.

2. The bottom of the footing should be placed at the same level as that of the adjacent machinery foundation so that deeper excavation alongside will not weaken the soil under either one. The column foundation will probably be built before the machinery foundation.

3. The steel base plate is to be raised above the floor. Therefore, a concrete pedestal is necessary, and its top should be as small as practicable

in order to minimize the obstruction around the column and above the floor.

4. The floor slab is to be seated upon the pedestal. Therefore, a 3-in. shelf is desirable around the pedestal.

5. The footing should be symmetrical, but one side dimension should not exceed 7 ft. because of the adjacent machinery foundation.

The minimum area of the footing should be slightly larger than 275/4 = 69 ft.^{2}; 7 by 10 ft. will be used as the trial dimensions of the footing.

The top of the pedestal will be made 2 ft. 6 in. square and provide an area of concrete in bearing that is 2.5 times that of the base plate. The main shaft of the pedestal is therefore 3 ft. square.

How thick should the footing be? Since the large pedestal may eliminate the seriousness of punching shear, the bending and the longitudinal shear in the long direction will probably control. A reasonable guess for footings with heavy pressures is a cantilevered projection from the pedestal of one to two times the effective depth of the footings. For smaller bearing pressures, like this one, the projection may be 1.5 to 2.5 (or perhaps 3) times the effective depth. The maximum projection is 3 ft. 6 in.; therefore try d = 18 in., with a cover of 3 in. over the rods. A sketch of the footing is shown in Fig. 5-18. Assume that the stock reinforcement to be used is ½- and ¾-in. round rods.

What is the best method to use in the analysis of such a footing? Obviously, the bending in the long direction will be the larger. This might be computed on the rectangular assumption, using area $PKLU$ of Fig. 5-18(c). For one interpretation of the trapezoidal method, the area $QKLR$ might be assumed. However, for elongated footings, it seems more sensible to assume that lines RX and QY of Sketch (c) be drawn at 45° from line QR extended. Then assume that the full pressure under area $QYKLXR$ acts about the axis QR, and the pressure under area $RXZS$ acts similarly about RS in the short direction. It is obvious that, for long and narrow footings, a trapezoid shaped like $RLMS$ seems to be less reasonable than $RXZS$ when considering bending across the short direction. This modified trapezoidal section will therefore be used.

The assumed increase of pressure caused by the footing itself is

$$\Delta P \text{ for the footing} = (7 \times 10 \times 1.75 + 3^2 \times 2.58)50 = 7{,}300 \text{ lb. (approx)}$$

$$\text{Total } p = \frac{275 + 7.3}{70} = 4.03 \text{ k.s.f. (near enough)}$$

$$\text{Effective } p = \frac{275}{70} = 4 \text{ k.s.f. (approx.)} \quad \text{This is used for analysis.}$$

$$v_T = \frac{(70 - 9)4{,}000}{4 \times 36 \times 0.3 \times 18} = 314 \text{ p.s.i.} \quad \text{(approx, but much less than 600 allowed)}$$

Long direction:

The shear and the moment about QR for area $QYKLXR$ are found as follows:

Part	Area, ft.^{2}	Lever arm	Moment of area, ft.^{3}
Large rectangle. . .	$7 \times 1.5 = 10.5$	2.75	28.9
Small rectangle. . .	$3 \times 2 = 6.$	1.	6.
Two triangles. . . .	$\dfrac{2 \times 2 \times 2}{2} = 4.$	1.33	5.3
	Total = 20.5		Total = 40.2

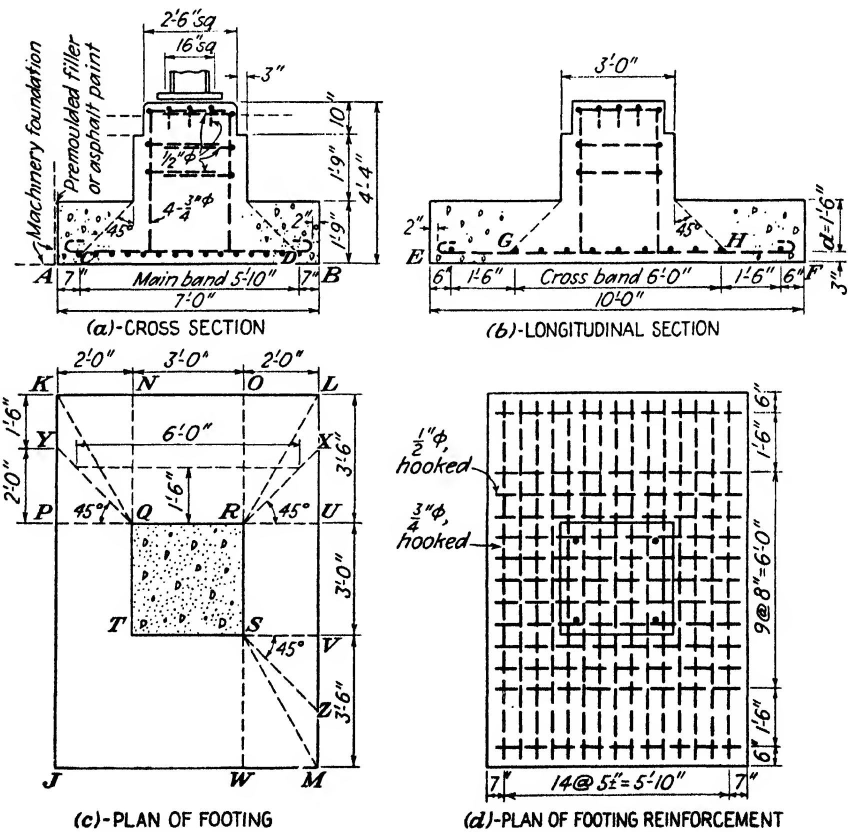


FIG. 5-18. A spread footing to be used at an industrial plant.

Therefore,

$$V_{QR} = 20.5 \times 4 = 82 \text{ kips}$$
$$M_{QR} = 40.2 \times 4 = 161 \text{ ft.-kips}$$
$$A_s = \frac{M}{f_s jd} = \frac{161}{18 \times 0.9 \times 1.5} = 6.62 \text{ in.}^2 \text{ required}$$
$$f_c = \frac{161{,}000 \times 2 \times 12}{0.3 \times 0.9 \times 72 \times 18^2} = 610 \text{ p.s.f.}$$
$$\Sigma o = \frac{V}{ujd} = \frac{82{,}000}{170 \times 0.9 \times 18} = 29.8 \text{ in.}^2 \text{ required}$$

Use fifteen ¾-in. round rods at 5 in. c.c. ($A_s = 6.6$, $\Sigma o = 35.4$)

$$\text{Diagonal tension } v_L = \frac{4{,}000\,[7 \times 1.5 + \left(\frac{6+7}{2}\right)0.5]}{72 \times 0.9 \times 18} = 47 \text{ p.s.i. (90 allowed)}$$

Short direction:

Part	Area, ft.²	Lever arm	Moment of area, ft.³
Rectangle........	$2 \times 3 =$ 6.	1.	6.
Two triangles.....	$\frac{2 \times 2 \times 2}{2} =$ 4.	1.33	5.3
	Total = 10.		Total = 11.3

Therefore,

$$V_{RS} = 10 \times 4 = 40 \text{ kips}$$
$$M_{RS} = 11.3 \times 4 = 45 \text{ ft.-kips}$$
$$A_s = \frac{45}{18 \times 0.9 \times 1.5} = 1.8 \text{ in.}^2 \text{ required}$$
$$\Sigma o = \frac{40{,}000}{170 \times 0.9 \times 18} = 14.5 \text{ in.}^2 \text{ required}$$

Use ten ½-in. round rods at 8 in. c.c. ($A_s = 2.0$, $\Sigma o = 15.7$).

v_L is automatically less than in the long direction and need not be checked. This footing will be accepted, and it is drawn in more detail in Figs. 5-18(*a*), (*b*), and (*d*). The reader should realize that this is not the only footing that will serve the required purpose. In a large project there may be many footings, and it may be desirable to prepare designs for several different sizes and use them whenever they are safe and not too wasteful.

5-8. Combined footings. There are conditions under which it is advisable to use a large slab or combined footing under two or more columns instead of an isolated spread footing under each one. Some such conditions are illustrated in Fig. 5-19:

1. When a row of columns is to be so close to a building line that spread footings would be loaded eccentrically. A common footing may be placed under each outer column and the corresponding one in the adjoining interior row, as pictured in Sketch (a).

2. When two columns are so close together that isolated footings would practically coincide; *e.g.*, footing A in Sketch (b). The footing under a

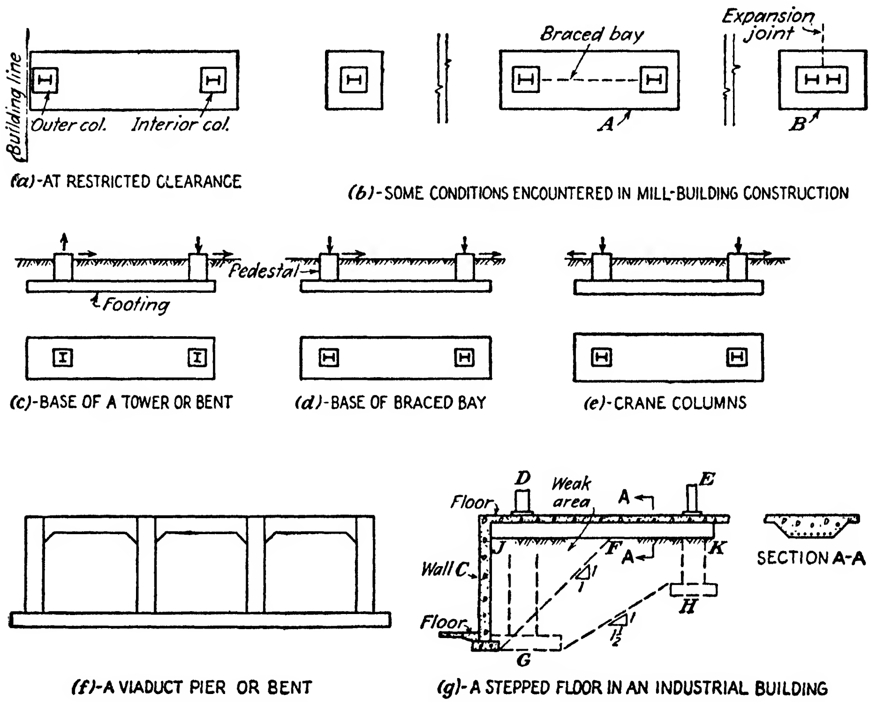


FIG. 5–19. Some conditions where combined footings may be advantageous.

double column at an expansion joint in the superstructure, as shown by B, is practically an isolated footing rather than a combined one.

3. When the designer wishes to tie two footings together to resist uplift, overturning, and opposing horizontal forces, as illustrated in Sketches (c), (d), and (e).

4. When such a structure as a bridge or viaduct pier composed of two or more columns rests upon soil of low bearing value or upon piles, so that it is desirable to have one large common base, as shown in Sketch (f).

5. When the principle of combined action seems to offer the most desirable construction. The special conditions encountered in engineering practice vary amazingly, and each design should be the best for that particular case.

For example, Sketch (*g*) illustrates a situation that was encountered in an industrial plant located on a hillside. The floors were placed at different levels. The position of wall *C* was fixed by the machinery, whereas the column locations were determined upon the basis of the preferable structural framing. Because of the probable weakening of the soil in area *F* on account of the excavation for the lower floor, it would be desirable to place the footing for the heavy column *D* down at some such position as shown by the dotted lines at *G*. This excavation would, in its turn, make it advisable to build the footing of the lighter column *E* at approximately the dotted location *H*. This construction would involve considerable cost for excavation and substructures and would require that the foundations of the heavy machines be extended down to undisturbed soil. Even the fact that portions of the concrete floor would be placed upon soil that was disturbed in some cases, firm in others, and backfilled around the columns would be disadvantageous because of the uneven settlements that would occur unless the floor, too, were structurally supported.

It was therefore decided to use a strong combined footing or beam under both columns. This was a deepened portion of the floor built integrally with the adjacent floor slab. The footing and the floor slab were seated upon the wall at *J*. The load from column *D* was thus spread across to *J* and to the undisturbed earth between *F* and *K*. The load from column *E* was included in the forces affecting the soil *FK*. This scheme proved to be satisfactory.

The planning of a combined footing requires that the designer determine how he is going to "make" the structure act. Undoubtedly, the footing will behave as an entity that supports the loads as efficiently as it can, and the actual stresses may be unknown. However, it is the safety of the structure that is important rather than a knowledge of the magnitudes of the stresses in it. If the designer provides a definite, adequate, and efficient system that is capable of resisting all the anticipated forces and if he has anticipated them wisely and well, the structure will behave substantially as he intended before it will fail. It is to be understood that the engineer will provide a system that will be as simple, direct, and positive in its action as he can devise.

Several designs and analyses of combined footings are made in this chapter and following ones. By studying them, the reader will learn how to attack such problems.

Example. In Fig. 5-20 is shown the lower end of an assumed two-column bent for a reinforced-concrete viaduct. The columns are close to a building line that limits the width of the footings along one side, although an overhead easement has been secured for the superstructure. The engineer in charge has tested the soil and limited the allowable bearing pressure for vertical loads to 2 tons per ft.^{2} Although the column loads differ because of the weight of a cantilevered sidewalk on one side of the superstructure, the columns and pedestals are to appear alike. Design and detail a substructure to support this viaduct, keeping the edge of the concrete at least 6 in. back from the building line.

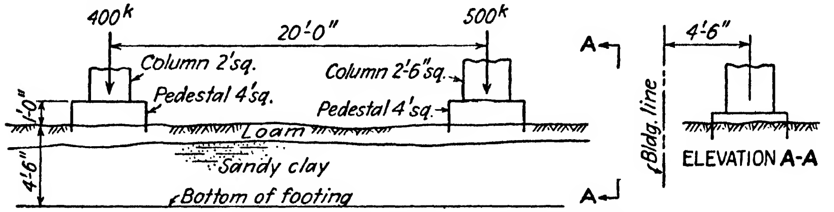


FIG. 5-20. Load diagram and column dimensions for a viaduct bent.

If isolated spread footings are used, the approximate area of the larger one will be

$$A = \frac{500 + 16}{4} = 129 \text{ ft.}^2$$

where the 16 kips is a guess at the added load caused by the footing itself. A square footing would thus be nearly 11 ft. 6 in. on a side; a rectangular one 8 ft. wide would be 16 ft. long. It therefore seems that a combined footing may be preferable to separate ones.

The total area of a combined footing may be, approximately,

$$A = \frac{900 + 30}{4} = 232 \text{ ft.}^2$$

Therefore, try a symmetrical footing as shown in Fig. 5-21(*a*).

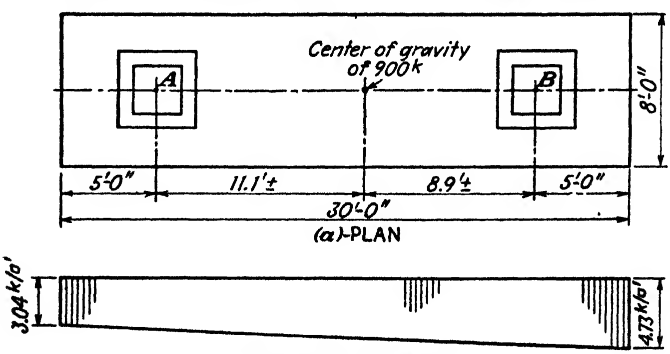


FIG. 5-21. A symmetrical, combined footing.

The center of gravity of the loads of the columns is located 1.1 ft. away from the center of the footing. Using this figure, since the effect of the footing's weight is relatively negligible, the soil pressures at the ends are found as follows, assuming uniform variation in intensity:

$$p_{max} = \frac{P}{A}\left(1 + \frac{6e}{L}\right) = \frac{930}{8 \times 30}\left(1 + \frac{6 \times 1.1}{30}\right) = 4.73 \text{ k.s.f.}$$

$$p_{min} = \frac{930}{240}\left(1 - \frac{6 \times 1.1}{30}\right) = 3.02 \text{ k.s.f.}$$

This, as foreseeable at a glance, is unsatisfactory because of the excessive pressure near one end.

How may the footing be shaped to secure uniform pressure? Although not applicable in this case, a shape like that pictured in Fig. 5-22 might be used. The trial dimensions TD, DL, and MN are chosen, then NP is computed as follows upon the basis that the centers of gravity of the bearing area and the loads should approximately coincide:

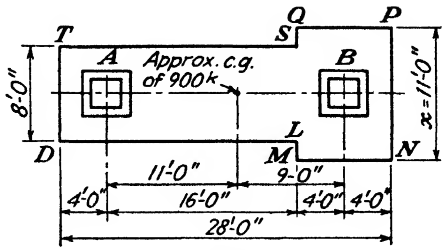


FIG. 5-22. An unsymmetrical, combined footing.

$$8 \times 15 \times 7.5 = 8 \times 5 \times 2.5 + 8 \times 9 \times NP$$
$$NP = 11.1 \text{ ft. (call it 11 ft.)}$$

$$p = \frac{930}{8 \times 20 + 8 \times 11} = 3.75 \text{ k.s.f.}$$

This pressure is slightly below the allowable, and the footing might be reduced a little. However, the proximity of the building line in this case prevents the use of this sideward spreading of loads.

Another method is to use a rectangular footing like that of Fig. 5-21 but relocated so that its center coincides, at least approximately, with the center of gravity of the loads, as shown in Fig. 5-23(*a*). A trial footing is then drawn as in Fig. 5-23(*b*). Because of the probable large upward bending moment in the central portion of the footing, the two pedestals will be joined by a 4-ft. rib, forming an inverted T-beam, thus obtaining considerable depth without the use of an excessive amount of concrete.

Neglecting the weight of the footing, the pressure on its bottom is

$$\frac{900}{30} = 30 \text{ kips per lin. ft.}$$

The extra applied weight of the footing in excess of the weight of the earth is, approximately,

$$(8 \times 30 \times 1.5 + 4 \times 24 \times 3)0.05 = 32 \text{ kips}$$

Therefore,

$$\text{Total } p = \frac{932}{240} = 3.9 \text{ k.s.f. (satisfactory)}$$

With a uniform load of 30 kips per lin. ft. and point loading at the columns, the longitudinal shear diagram is as shown in Fig. 5-23(c); the bending-moment diagram, in (d). The ordinates are approximately but not technically correct because the center of the footing is to be placed 0.1 ft. from the point C of Fig. 5-23(a) in order to avoid unimportant refinement of dimensions. Although the projection of the pedestal above the rib is small, it will be satisfactory to assume that the column loads are spread uniformly over the areas of the pedestals. This reduces the theoretical bending moments under the columns as shown by the dotted lines in Fig. 5-23(d), and these values will be used for design purposes: 85 ft.-kips near A and 315 ft.-kips near B.

The computations of the bending moments for which the projecting parts at F and G are to be proportioned will omit the 0.85 coefficient used in similar computations for isolated footings. The allowable bond unit stress will be limited to $0.05f'_c = 150$ p.s.i. as for ordinary beams, since this footing is primarily a one-way beam.

The determination of the necessary reinforcement at critical points and the analysis of the trial footing are as follows, referring to Fig. 5-23:

$$A_s \text{ at } K = \frac{1{,}070}{18 \times 0.9 \times 4.25} = 15.5 \text{ in.}^2$$

For convenience, the units used are foot-kips and feet. If ten $1\frac{1}{4}$-in. square rods are used, $A_s = 15.6$ and $\Sigma o = 50$.

To check the critical bond stress on these rods, use the most rapid change in bending moment—between O and J—in Sketch (d), and see if the rods can develop the necessary resistance fast enough.

$$u - \frac{\text{change in } M \text{ per lin. in.}}{\text{surface area of rods per lin. in.} \times jd}$$

$$u = \frac{800{,}000}{(4 \times 12)50 \times 0.9 \times 4.25} = 87 \text{ p.s.i.}$$

This is a low value and shows that part of the rods can be discontinued near H and J if desired.

$$f_c \text{ at } K = \frac{2 \times 1{,}070{,}000 \times 12}{0.3 \times 0.9 \times 76 \times 51^2} = 480 \text{ p.s.i.}$$

when the effective width is assumed to be the width of the rib plus $2d$ of the footing.

$$A_s \text{ at } G = \frac{240}{18 \times 0.9 \times 1.25} = 11.9 \text{ in.}^2$$

$$\Sigma o \text{ at } G = \frac{120{,}000}{150 \times 0.9 \times 15} = 59.3 \text{ in.}^2$$

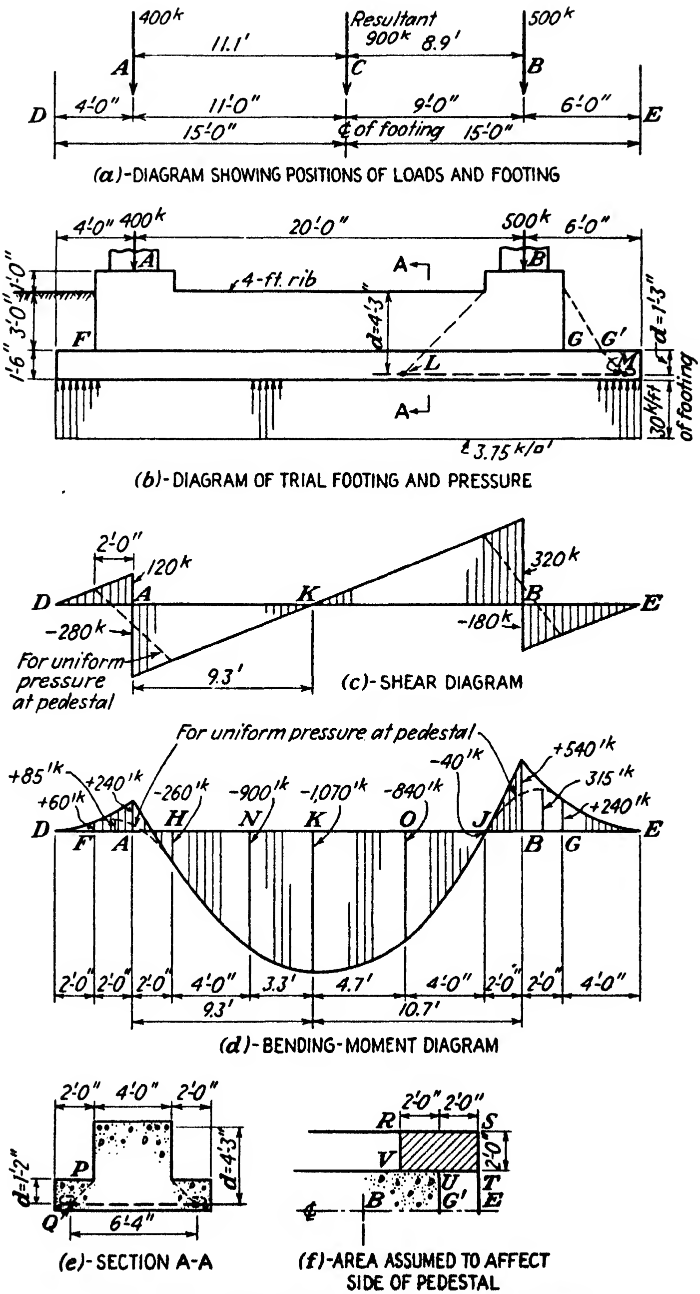


FIG. 5-23. Analysis of a trial, ribbed, combined footing.

Below B, neglecting the shallow top of the pedestal, the required steel may be assumed to equal

$$A_s = \frac{315}{18 \times 0.9 \times 4.25} = 4.6 \text{ in.}^2$$

The surface area required for bond per linear inch to cause the change of bending moment from B to J, Sketch (d), is

$$\Sigma o = \frac{\text{change in } M \text{ per lin. in.}}{\text{allowable bond stress per lin. in.} \times jd}$$

$$\Sigma o = \frac{315{,}000}{(2 \times 12)150 \times 0.9 \times 4.25} = 23 \text{ in.}^2$$

Obviously, the bending at the section through G will control the reinforcement, and at least twenty ⅞-in. round rods at 6 in. c.c. will be needed.

$$f_c \text{ at } G = \frac{2 \times 240{,}000 \times 12}{0.3 \times 0.9 \times 76 \times 15^2} = 1{,}250 \text{ p.s.i. (rather high)}$$

A careful examination of Fig. 5-23(b) and a consideration of the results of the computations of A_s and f_c at G show that this point G is a weak spot in the design. It will be remedied easily by sloping the end of the pedestal or rib at B as shown by the dotted line to G' in this sketch. This will make the projecting portions of the slab at both ends alike. Therefore,

$$A_s \text{ at } G' = \frac{60}{18 \times 0.9 \times 1.25} = 3 \text{ in.}^2$$

$$\Sigma o \text{ at } G' = \frac{60{,}000}{150 \times 0.9 \times 15} = 30 \text{ in.}^2$$

It will be satisfactory to use eleven ¾-in. round rods at 8 in. c.c. in order to meet the requirements already found at the section near B. The reinforcement near A will be made the same as at B. Furthermore, f_c will now be small at F and G' and need not be recomputed.

For the side projections of the footing,

$$A_s \text{ at } P = \frac{3.75 \times 2 \times 1}{18 \times 0.9 \times 1.17} = 0.4 \text{ in.}^2$$

$$\Sigma o \text{ at } P = \frac{3{,}750 \times 2}{150 \times 0.9 \times 14} = 4 \text{ in.}^2$$

Therefore, ½-in. square rods may be used at 6 in. c.c.

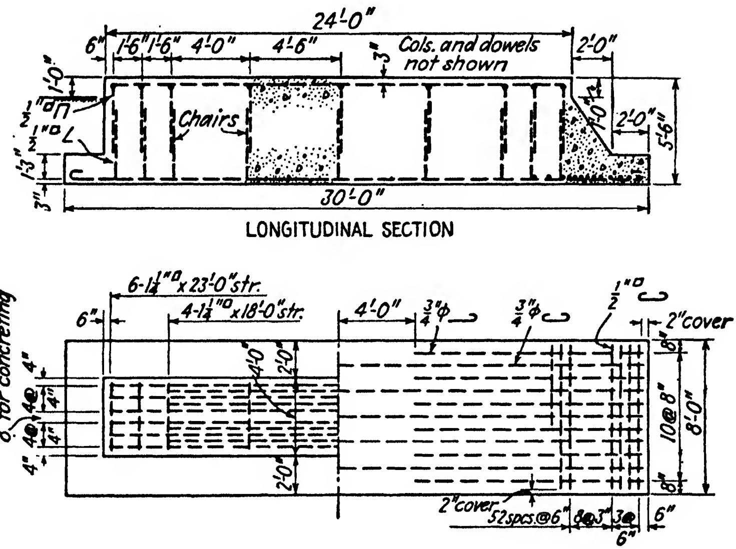
Assuming that a band of steel will cross an arbitrarily chosen 2-ft. strip VU of Sketch (f) at the ends of the deep rib and that this will be able to withstand the bending caused by the pressure under the area $RSTV$,

$$V_{VU} = 3.75 \times 2 \times 4 = 30 \text{ kips}$$

$$M_{VU} = 30 \times 1 = 30 \text{ ft.-kips}$$

$$A_s \text{ for } VU = \frac{30}{18 \times 0.9 \times 1.17} = 1.6 \text{ in.}^2$$

$$\Sigma o \text{ for } VU = \frac{30{,}000}{150 \times 0.9 \times 14} = 16 \text{ in.}^2$$



HALF PLAN OF TOP MAT PARTIAL PLAN OF BOTTOM MAT

FIG. 5-24. Details of ribbed footing.

Therefore, use ½-in. square rods at 3 in. c.c. across this strip.

The critical shearing unit stresses probably will not exceed the following:

$$v_T \text{ at } F \text{ and } G' = \frac{2 \times 30{,}000}{48 \times 0.3 \times 15} = 280 \text{ p.s.i.}$$

$$v_T \text{ at } J = \frac{320{,}000 - 2 \times 30{,}000}{48 \times 0.3 \times 51} = 350 \text{ p.s.i.}$$

$$v_L \text{ at } M = \frac{30{,}000 \times 0.75}{76 \times 0.9 \times 15} = 22 \text{ p.s.i.}$$

$$v_L \text{ at } L = \frac{320{,}000 - 6.25 \times 30{,}000}{48 \times 0.9 \times 51} = 60 \text{ p.s.i.}$$

All these calculations indicate that the thickness of the footing might be reduced slightly. The central rib might be narrower also, but it is easier to construct if it is the same width as the pedestals. Such changes will not cause much saving of money. Therefore, the trial design will be accepted as far as the ribbed type is concerned, and the steel will be arranged as shown in Fig. 5-24.

Another possible design for a footing to be used for the conditions in Fig. 5-20 is pictured in Fig. 5-25(a). The footing is made narrower than the one in Fig. 5-23 so that the cantilevered ends beyond F and G will cause more restraint and reduce the bending moment in the central portion K. This is done in order to eliminate the necessity of a central rib, thus permitting

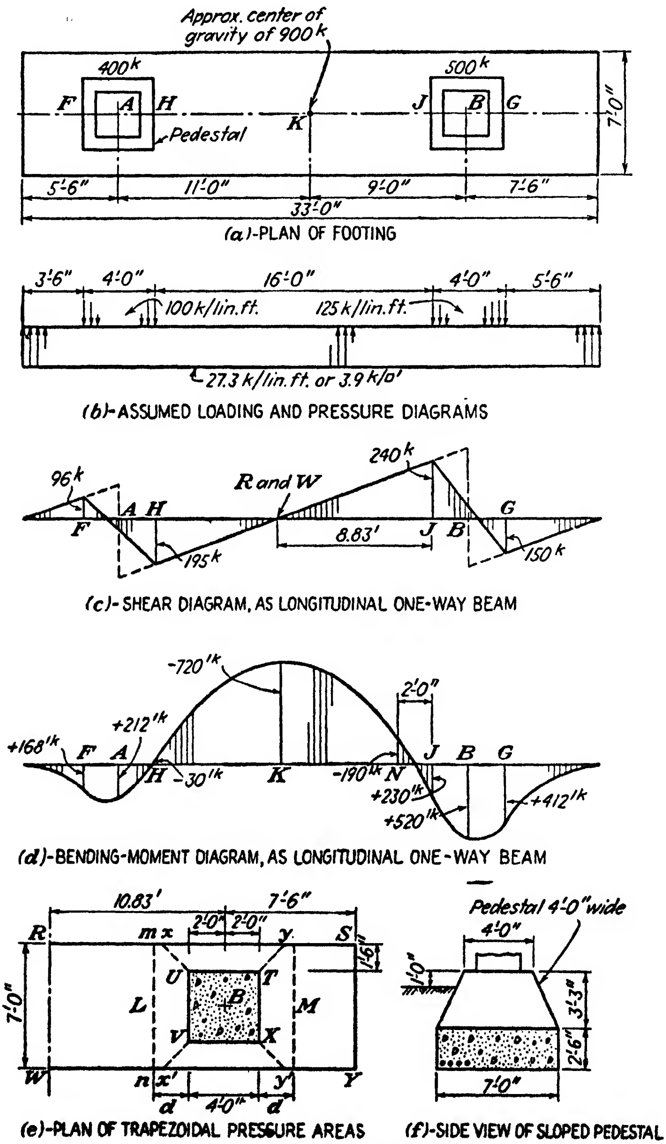


FIG. 5-25. A combined slab-and-pedestal footing.

the use of a heavy simple footing. The footing is primarily a one-way beam.

The shear and bending-moment diagrams for this footing are shown in Figs. 5-25(*c*) and (*d*). The pedestals are assumed to spread the column loads uniformly.

The magnitudes of the bending moments under the pedestals are really questionable. In the case of a symmetrical isolated footing, the bending moment was considered to be constant across the bottom of the pedestal. This was permissible because the opposite sides were balanced so that rotation of the column did not occur. In this combined footing, however, this assumption does not seem to be valid.

For example, Fig. 5-26 shows the shear and bending-moment diagrams when the footing under the pedestal is assumed to be rigidly fixed. Sketch (*b*) indicates that the bending moments at F and H, and at G and J, are unequal. This fixed condition is improbable. Unless the columns are very stiff, they will rotate until they conform with whatever slope the footing under them takes when it is in equilibrium. It is also obvious that the assumption of point loading at the columns gives results that are unreasonably large for the bending moments at the columns. It therefore seems desirable to adhere to the principles of the calculations used in computing the ordinates in Fig. 5-25(*d*). Continuity and frame action of the columns and the footing combined may be considered when the columns are so stiff that their resistance to rotation becomes important. Generally, however, the columns are relatively flexible.

The maximum shear in this footing is at J, but the area of effective resistance to transverse shear is questionable. Some broad assumptions must be made. Therefore, referring to Fig. 5-25(*e*), the following procedure might be adopted for such cases:

1. Let RW represent the point of zero shear in Sketch (*c*).
2. From the corners of the pedestal draw lines at 45° to the sides of the footing, as shown by the dotted lines Ux and Ty.
3. At first, assume that the pressure under the area $RxUVx'W$ causes shear on UV, and that the pressure under the area $SyTXy'Y$ causes shear on TX.
4. Similarly, assume that the pressure under the areas $yTUx$ and $x'VXy'$ causes shear on TU and VX, respectively.
5. If the estimated force on UV greatly exceeds that on face UT or VX, as it obviously does here, there is a general tendency for the whole width of the footing in the plane UV to resist shearing. The projecting parts beyond U and V tend to transfer the shears around the corners to the adjacent portions of sections UT and VX. In such a case, compute the shear on UV as though it alone resisted the force applied by the pressure under area $RxUVx'W$, then temper the result by using good judgment as to how excessive the computed stress may be.

6. Use these previously designated areas for estimating the diagonal tension beyond any corresponding face of the pedestal. For example, at L of Fig. 5-25(*e*), assume the effective width to be that of the section across the assumed area at a distance d from the face of the pedestal.

7. Use the computed shear at such an edge as UV to estimate the bond unit stress on the rods perpendicular to that edge.

In order to obtain some ideas for assuming a trial depth, compute the approximate minimum permissible depths for shearing resistance at UV and mn of Fig. 5-25(*e*). Therefore, compute first the assumed pressure under area $RxUVx'W$.

$$V_{UV} = \left(7 \times 8.83 - \frac{2 \times 1.5 \times 1.5}{2}\right) 3{,}900 = 232{,}000 \text{ lb.}$$

$$d = \frac{V}{v_T b k} = \frac{232{,}000}{600 \times 48 \times 0.3} = 27 \text{ in.}$$

Then test the diagonal tension at section mn on the basis of this thickness. Line mn is beyond x and x', hence the pressure under area $RmnW$ is

$$V' = 7 \times 6.58 \times 3{,}900 = 180{,}000 \text{ lb.}$$

$$v_L = \frac{V}{bjd} = \frac{180{,}000}{84 \times 0.9 \times 27} = 88 \text{ p.s.i.}$$

It therefore appears that an effective depth of 27 in. is the minimum desirable. This will be tried. The areas of reinforcement are to be determined, and the compressive strength should be checked.

$$A_s \text{ at } K = \frac{720}{18 \times 0.9 \times 2.25} = 20 \text{ in.}^2$$

Since the largest bending moment is at the center of the footing, this is the critical point for compression in the concrete. Therefore, instead of assuming $k = 0.3$, use Fig. 1 in the Appendix and find $k = 0.34$, using $n = 10$ and $p = 20/(84 \times 27) = 0.0088$. Then $j = 1 - \frac{1}{3}k = 0.89$. Therefore,

$$f_c \text{ at } K = \frac{2 \times 720{,}000 \times 12}{0.34 \times 0.89 \times 84 \times 27^2} = 930 \text{ p.s.i. (satisfactory)}$$

The maximum bond stress on the basis of the change in bending moment will occur between J and N, Fig. 5-25(*d*). Thus the required surface area of the reinforcement is

$$\Sigma o = \frac{\text{change in } M \text{ per lin. in.}}{(jd)u}$$

$$= \frac{420{,}000 \times 12}{24 \times (0.9 \times 27) \times 150} = 58 \text{ in.}^2$$

Assuming that the pedestal will be poured after the footing has set and that the longitudinal shearing strength at the junction is not sufficient to cause the pedestal to participate effectively in the beam action of the footing, the effective depth under the pedestal is assumed to equal that of the footing alone. Therefore,

$$A_s \text{ at } B = \frac{520}{18 \times 0.9 \times 2.25} = 14.3 \text{ in.}^2$$

Using the shear as computed from area $RxUVx'W$, Fig. 5-25(e),

$$\Sigma o \text{ at } UV = \frac{232{,}000}{0.9 \times 27 \times 150} = 64 \text{ in.}^2$$

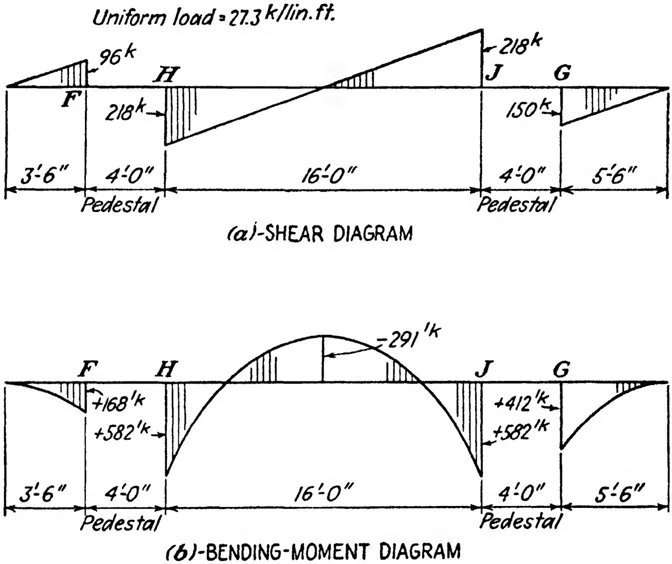


FIG. 5-26. Improbable shear and bending-moment diagrams, assuming ends fixed at pedestals.

At A, the same requirements as at B will be assumed because the bond resistance must be practically the same at both places.

The transverse reinforcement in the footing under and near the pedestals is important. Elsewhere it serves as ties only. Assuming that the pressure under the area $xUTy$ of Fig. 5-25(e) will cause transverse bending about UT as in an isolated footing, then

$$M_{UT} = 3.9\left(4 \times 1.5 \times 0.75 + \frac{2 \times 1.5 \times 1.5 \times 1}{2}\right) = 26.3 \text{ ft.-kips}$$

$$A_s = \frac{26.3}{18 \times 0.9 \times 2.17} = 0.75 \text{ in.}^2$$

$$\Sigma o = \frac{3{,}900\left(\frac{4+7}{2}\right)1.5}{0.9 \times 26 \times 150} = 9.2 \text{ in.}^2$$

This reinforcement will be placed within the width xy of Sketch (e), and the same amount will be used under the other pedestal. If this transverse bending and the punching shear were objectionably large, the sides of the pedestal might be sloped as shown in Fig. 5-25(f), thus enabling the main footing to act entirely as a beam that is reinforced in one direction.

TABLE 5-1. Comparison of Costs of Two Combined Footings

Type	Item	Quantity	Unit price	Cost of item
Fig. 5-24	Concrete	28 cu. yd.	\$25 per cu. yd.	\$700
	Forms	340 ft.^{2}	\$0.20 per ft.^{2}	68
	Reinforcement	2,280 lb.	\$0.07 per lb.	160
	Total.....	....	....	\$928
Fig. 5-27	Concrete	25 cu. yd.	\$25 per cu. yd.	625
	Forms	304 ft.^{2}	\$0.20 per ft.^{2}	61
	Reinforcement	4,300 lb.	\$0.07 per lb.	301
	Total.....	....	....	\$987

A drawing of the reinforcement adopted for this footing is given in Fig. 5-27. A comparative estimate of the footings of Figs. 5-24 and 5-27 is given in Table 5-1. The unit prices are to be considered as applicable for

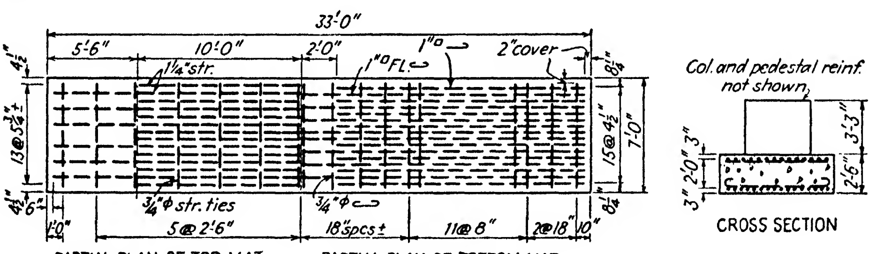
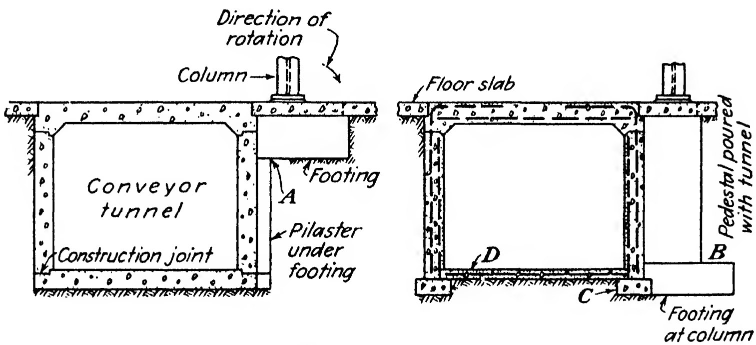


FIG. 5-27. Reinforcement for combined slab-and-pedestal footing.

comparison only because costs will vary in different places and for different conditions. It is assumed that the ground will be excavated by mechanical equipment and that the sides of the cut will be sloping so that edge forms will be required for the footings. From these computations as shown in Table 5-1, it seems that the footing of Fig. 5-24 will be slightly more economical than the other.

5-9. Special combinations. Some illustrations of special construction are shown in Figs. 5-28 to 5-30. The purpose of each is to show a design or plan for a foundation that was used to support a structure that could not be supported very well on ordinary isolated footings. The drawings are greatly simplified in order to emphasize the principles involved rather than the details. However, the main reinforcement is shown by dotted lines so that the reader may see what paths of resistance were provided.

When planning such special construction, the designer should approach each problem with an open mind. He should find out what the existing conditions are, what is wanted, and how he may accomplish it. Amazing are the instances of an individual's reliance upon precedent—what was done on some other job. Too often he does not know the reasons for the other construction, and these may be very different from those that should affect his problem. Creative engineering thinking, a thorough knowledge of



(a)-COLUMN FOOTING ON "HARD SPOT" AT ONE SIDE-UNWISE CONSTRUCTION (b)-COLUMN FOUNDATION EXTENDED DOWN TO FIRM GROUND-BETTER CONSTRUCTION

FIG. 5-28. Column foundation when near edge of excavation.

structural action, and a keen sense of practicality—these are attributes that he should possess and exercise instead of thumbing through books and looking over blueprints in search of a design already prepared by his predecessors.

In Fig. 5-28(a) is shown a condition that occurred in a machine shop where a conveyor tunnel was to run under a building and close to one line of crane columns. If the footing for a column were made separate from the tunnel, it would have an objectionable eccentricity. If it were supported along one edge of the tunnel by a seat at A, it would rest partly upon backfilled or disturbed soil so that it would probably settle and rotate about the seat as an axis. Therefore, the column foundation was incorporated in the conveyor tunnel construction, as shown in Sketch (b). The footing B was extended sufficiently to prevent any serious tipping because of the large resistance of the soil under the long narrow bearing at C under the tunnel wall. The floor D was poured later so as to avoid its participation in the original distribution of the heavy dead loads.

Figure 5-29(a) shows the typical foundation used under one pair of columns for a long elevated coalbin that was incorporated in an extension of a boiler house. Column A had to be close to the existing wall because of the coal chutes and aisle clearances. The additional load could not be supported by the existing wall and its footing; neither should this footing be cut off on one side to permit the new construction to encroach upon it.

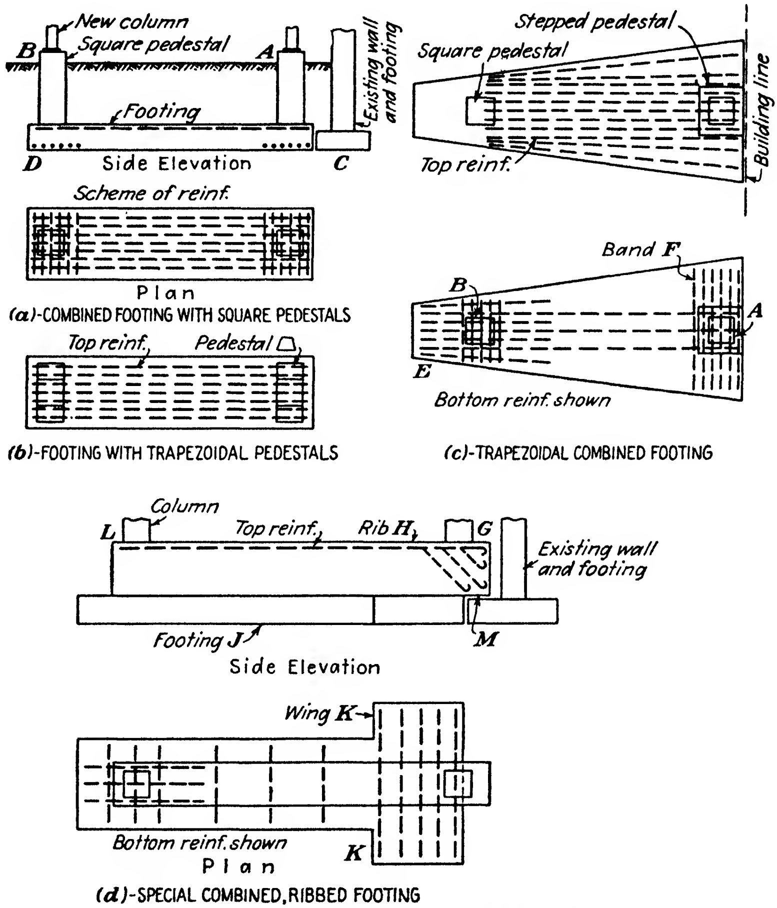


FIG. 5-29. Examples of combined footings on weak soils.

A combined footing was therefore used. The pedestal for column A was set close to one end of the slab, the latter being isolated from footing C. Column B was set similarly with respect to end D in order to equalize the pressures under the footing. The footing was primarily a one-way beam with tension in its top, and the slab was thick enough to withstand the shears caused by the concentrated loads at the pedestals. If the loads had been sufficiently heavy, the pedestals might have been sloped as shown in Sketch (b), or a ribbed footing could have been used.

Sometimes, in a situation like that of Fig. 5-29(*a*), the footing may be shaped as in Sketch (*c*), where end *E* is cantilevered in order to cause restraint under column *B* and thereby reduce the tension in the top of the slab. The dimensions are chosen so as to keep the center of gravity of the footing under the resultant of the applied loads. The reinforcing band *F* acts somewhat as in an isolated footing. This shape of footing is also useful when the load on column *A* is much larger than that on *B*, and even when *B* exceeds *A*.

Still another case is illustrated in Fig. 5-29(*d*). An extension was built alongside a building, but the existing framing and foundations could not support the added loads. A new row of columns *G* was required as close to the old wall as possible in order to support the new framing and provide a vertical deflection joint at the junction of the new and the old structures. A large ribbed footing was therefore used under *G* and *L*, the latter column serving as an anchor for the cantilever. In order to obtain reasonably uniform pressures, footing *J* was made with the wings *K*. A clearance of 2 in. was provided at *M* in order to have space for the small but inevitable settlement that would occur as the soil was compressed by the new structure.

If combined footings, like *A* of Fig. 5-30(*a*), are used under a structure that is close to building lines or other obstructions, special construction is required at the corner *B*. One possible construction to hold the corner column is that used in this case, where a pair of combined footings join *B* with *C* and with *D*, each footing being sufficient to transfer its share of the load back onto adjacent adequate areas. Each footing resists the tendency of the other to twist under the eccentric load of *B*. Sometimes, heavy foundation walls serve this same purpose even better.

When combined footings are on plastic soil above sloping rock or gravel, as pictured in Fig. 5-30(*b*), harmfully unequal settlement should be prevented. It is possible to increase the area at *E* so that the bearing pressure is so much less there than at *F* that the estimated settlements of the footing are equalized. Another remedy is to increase the projection at *E* and decrease that at *F* to accomplish the same purpose. If the rock is sufficiently close and if the importance of the structure justifies the expense, it may be preferable to extend the pedestals to rock, as indicated at *J* and *K*.

At a large war plant, a deep pit was required close to crane columns, as pictured in Fig. 5-30(*c*). The pit was sunk as a caisson, then a heavy cantilevered beam *LM* was incorporated in the top of one side wall. Crane loads on the columns were eccentric with respect to the bottom *OQ*. However, the pit was so deep (over 50 ft.) that the abutting resistance of the earth on the side walls, such as *NQ*, was so great that no appreciable tilting could occur. If the pit had been shallow (perhaps 10 ft.), it would have been desirable to extend beam *N* to the bottom as a wall or as two pedestals joined to a thin side wall, and to have added a footing at Q, as shown by Sketch (d), in order to avoid too much eccentricity of load on the bearing area.

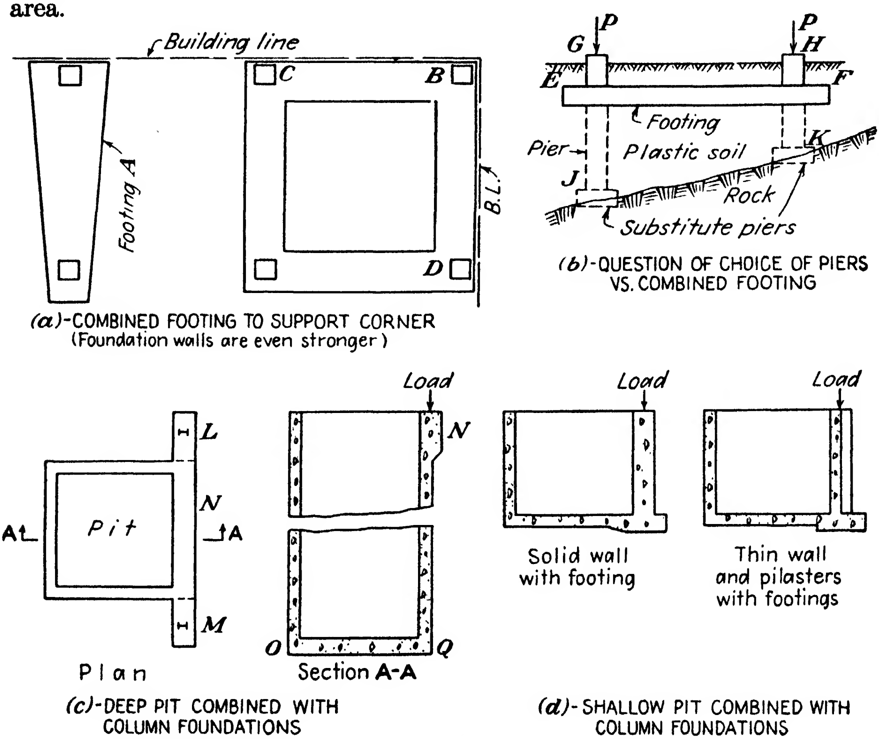


FIG. 5-30. Some examples of special foundations and footings.

PROBLEMS

5-1. A stepped footing like that of Fig. 5-18(a) has a pedestal 2 ft. 6 in. square and 3 ft. 6 in. high. It projects 6 in. above the ground but has no seat for the floor slab. The spread footing is 8 ft. square, 1 ft. 9 in. deep, with an effective depth of 1 ft. 6 in. The column reaction is 450 kips. Design the tensile reinforcement for this footing. Check the bond, shear, and diagonal tension in the concrete.

5-2. Assume a stepped footing with a pedestal exactly like that of Fig. 5-18(a). The main footing is 9 ft. square, is 27 in. deep, has 3 in. cover over the rods, and is to support a column load of 600 kips. Design the reinforcement and check the bond, shear, and diagonal tension.

5-3. Assume a footing exactly like that of Fig 5-18 except that the depth of the spread footing is increased 6 in. and the main rods are ⅞-in. rounds. The column load is 550 kips. Check the footing completely.

5-4. Assume a footing like that of Fig. 5-7(a). The column is 24 in. square and has a load of 350 kips. The footing is 7 ft. 6 in. square, is 24 in. deep, and its top is 3 ft. below the ground. The reinforcement is two perpendicular bands of thirteen ¾-in. rounds 6 in. c.c. with a cover of 3 in. Is the footing safe?

5-5. Design a footing like that of Fig. 5-7(*a*) to support a load of 300 kips if the soil is a rather soft silty clay and the frost depth is 5 ft. The column has a diameter of 30 in. Use square column having the same area.

5-6. Assume that a spread footing is similar to that of Fig. 5-7(*a*) but is 6 ft. wide and 8 ft. long. The column is 24 in. square and has a load of 320 kips. The footing is 1 ft. 9 in. deep. The lengthwise reinforcement is thirteen ¾-in. rounds at 5 in. c.c.; that across the narrow way is thirteen ¾-in. rounds at 6 in. c.c. centrally located. The cover over the rods is 3 in. Is this footing safe to hold a load of 320 kips?

5-7. Assume a sloped-top footing like that of Fig. 5-16(*c*) except that it is 8 ft. square, 1 ft. 6 in. thick at the edge, and the load is 380 kips. Is the footing safe in all respects?

5-8. Design a sloped-top footing to support a rectangular concrete column 28 × 32 in. having a load of 700 kips. The allowable bearing value of the soil is 6 k.s.f.

5-9. A sloped-top footing for a 36 in. square concrete viaduct pier has the following dimensions: width, 12 ft.; length, 14 ft.; thickness at center, 3 ft.; flat portion around pier, 6 in.; edge thickness, 2 ft.; cover over steel, 3 in.; reinforcement in long direction, twenty-three 1-in. rounds at 6 in. c.c.; reinforcement in short direction, seventeen 1-in. rounds at 8 in. c.c. through the center with one extra rod 8 in. from each end of the footing. Is this footing satisfactory if the load on the pier is 850 kips?

5-10. A footing for a steel column with a base plate 18 × 30 in. is limited to a width of 5 ft. parallel to the narrow side of the billet. The load on this crane column is 450 kips. The allowable bearing value of the soil is 3 tons per ft.^{2} Next to one of the narrow sides of the footing is to be a duct trench with its bottom 5 ft. below the floor. Design a footing to support this column.

5-11. Assume two concrete columns as in Fig. 5-20 except that both are 2 ft. 6 in. square and have loads of 480 kips each. They are spaced 18 ft. c.c. The bearing value of the soil is 4 k.s.f. Frost depth is 5 ft. Design a combined footing to support these two columns.

5-12. Two steel columns at an expansion joint in a mill building are 2 ft. 6 in. c.c. The base plates are 18 in. wide and 36 in. long. There is 12-in. clear space between the billets, and their bottoms are 6 in. above the floor. The column loads are assumed to be 300 kips each. The soil is a fine sand with thin lenses of inorganic silt. Design a combined or common footing for these columns.

5-13. Assume the columns, spacing, pedestals, and loads shown in Fig. 5-20. Then assume the footing of Fig. 5-22 to be used under these columns. Determine the thickness of concrete and the reinforcement needed for this footing.

5-14. A combined footing is to be used under three concrete columns 36 in. square. The columns are 15 ft. c.c. The load on the central column is 600 kips; that on each of the outer columns is 500 kips. The allowable bearing value of the ground is 3 tons per ft.^{2} Frost depth is 5 ft. Design a footing for these three columns.

5-15. A heavy machine weighing 750 kips is supported upon a base that is a hollow square 12 in. wide and 10 ft. square in outside dimensions. It rests upon a footing 18 ft. square and 24 in. deep. The reinforcement at the bottom consists of two perpendicular bands of seventeen hooked $1\frac{1}{4}$-in. square rods 12 in. c.c. with 3 in. of cover. No top reinforcement is used. Assume the pressure on top to be spread uniformly over the hollow square bearing area directly under the machine. The soil pressure is also assumed to be distributed uniformly. Is the footing satisfactory?

5-16. A brick chimney for a three-story house is 2×5 ft. at the basement floor. It weighs 30 kips. The soil is soft clay. Excessive settlement will crack the walls of the house. Design a footing for this chimney.

6

FOUNDATION WALLS

6-1. Ordinary simple walls. Houses, some industrial buildings, and many other light structures without basements may be founded upon shallow simple concrete walls. Their depth may depend upon the estimated penetration of frost below the ground's surface, the desired embedment beneath this surface, and the distance down to suitable soil, as illustrated in Fig. 6-1(*a*); the elevation of the top, upon the position of the top of the soil or of the ground floor of the structure. The thickness may be controlled by such matters as the thickness of the masonry or other wall resting upon the substructure, the minimum desired for pouring concrete, the total of the thickness of the supported wall plus the allowances for such details as a watertable and a seat for the floor construction. Some detailed sketches at the tops of foundation walls are given in Fig. 6-1.

The loads may be so small that firm sand and gravel will support the foundation wall and the superstructure without any footing, as indicated in Fig. 6-1(*b*). In other cases, a plain concrete footing may be used as in Sketch (*c*), and its width may be determined by the requirements of the building code, by the bearing value of the soil, or by good judgment. Heavy brick walls on weak soils may necessitate the use of wider footings reinforced as indicated in (*d*). However, in these simple cases of wall-bearing construction, the loads may be considered as uniformly distributed along the wall and across the footing unless truck doorways and other large irregularities occur that are sufficient to prevent the foundation wall from spreading the loads satisfactorily.

In cold climates it is generally desirable to extend foundation walls under truck doorways when the soil and moisture conditions are likely to cause frost heaving and subsidence of a concrete floor and driveway. An apron slab, as illustrated in Fig. 6-1(*f*), is advisable to span across any weak area near the wall. This combination will generally avoid interference with doors and cracking of pavement.

Shrinkage of concrete, caused by chemical action during curing, and shortening due to a decrease in temperature make it desirable to reinforce concrete foundation walls longitudinally. The amount of steel required is problematical. The chemical shrinkage occurs gradually but is practically completed in 4 to 8 weeks. A wall will try to shorten somewhat in accordance with its length, a figure of ⅜ in. in 100 ft. being useful for obtaining a scale on the probable deformation. This contraction is assumed to be uniform over the wall's cross section. Not only are contraction joints

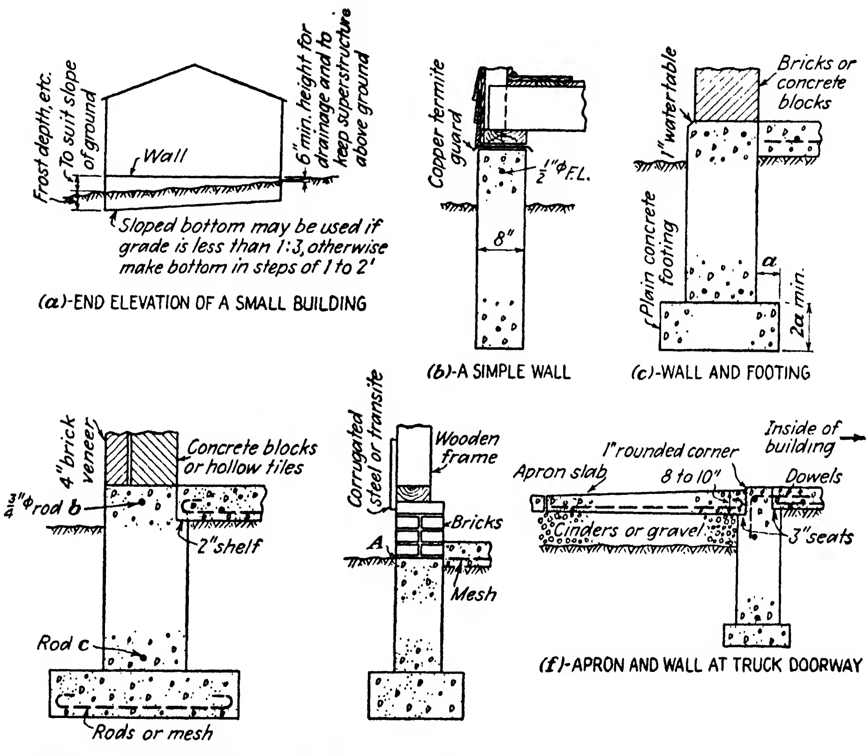


FIG. 6-1. Some examples of small foundation walls.

necessary, as explained in Art. 6-8, but the reinforcement should be sufficient to prevent localized visible cracks. A steel area 0.002 times that of the cross section of the concrete is usually sufficient for this purpose, and the reinforcement should be distributed evenly over the cross section, as indicated in Fig. 6-1(*d*).

When there is considerable surface area of a foundation wall exposed to wide and rapid variations of air temperature, thermal change may cause curvature or cracking of the wall. The longitudinal deformation may be a *total* variation in length of ¾ in. per 100 ft. per 100°F. change of temperature. For example, refer to Fig. 6-1(*d*) again. When the top of the wall is cooled quickly by the air, the top shrinks and rod *b* tends to distribute the deformation as hair cracks. In the meantime, the wall tends to curl upward near the ends because the bottom is not shortened so rapidly as the top. The weight and stiffness of the structure would probably prevent this curvature so that, were it not for rod *b*, one or two serious cracks might open up. Similarly, when the air temperature rises suddenly, the expansion of the top of the wall causes the wall to try to bow upward (convex on top) near the center. Rod *c* then resists the cracking that might occur when curvature is prevented. It is obvious then that reinforcement is needed near both the top and the bottom just as it is for resisting shrinkage. The steel area specified for shrinkage will generally resist the thermal effects as well. However, in extremely exposed situations, the reinforcement should be increased to 0.0025 or 0.003 times the cross section of the concrete.

Vertical reinforcement of small foundation walls may not be necessary when they are primarily compression members between the superstructure and the soil. A few rods may be useful as spacers and supports for the longitudinal steel, and they may be needed if anchor bolts cause sufficient uplift.

Foundation walls are sometimes built of concrete blocks, and even of bricks. Although it may be slightly more costly, poured concrete below ground is preferable, especially in cold wet climates. Brickwork at the surface of the ground at *A* in Fig. 6-1(*e*) is also likely to disintegrate. Any saving caused by a substitute for a good foundation is certain to be unimportant compared to dissatisfaction with the service performed by a structure that is not built properly.

Seldom is there need to calculate unit stresses in a foundation wall under a wall-bearing superstructure, except to determine the reinforcement needed in the footing, and for the bearing pressure upon the soil. There is, however, a very real opportunity for sensible design or planning.

Example. A businessman wishes to build a fire-resistant commercial garage. The walls are to be 12-in. brick unlined, the roof is to be precast concrete slabs with insulation and built-up roofing on top. The roof is to be supported by wall-bearing, steel bar joists or trusses 45 ft. long. The average estimated load per linear foot on the side walls is 4,100 lb. at the floor line. The floor level is to be 2 ft. above the ground for most of the wall, and the 6-in. reinforced-concrete floor slab is to be supported at its edges by the foundation wall. The probable frost depth is 3 ft.; the allowable bearing value of the soil, 2,500 p.s.f. Design a typical section of the foundation wall for this structure.

The first step in the planning is the determination of various structural features that are desired. For example, a 1-in. watertable is to be used because of appearance, a 2-in. shelf is to support the floor slab, the foundation is to be poured concrete, and the brickwork is to begin at the floor line. These features are pictured in Fig. 6-2.

The next step is to select the depth of the foundation. This is made 3 ft. 6 in. in order to be sure that it is below the danger line from frost.

The third step is to determine the necessary footing. Some allowance should be made for the fact that a truck wheel or other heavy load may be placed on the floor slab near B. Since the shelf CD holds the edge, the floor will have to support any load P and to span from CD to some area of

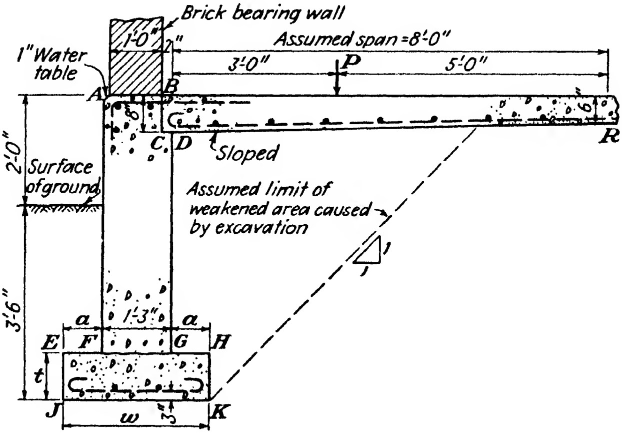


FIG. 6-2. Foundation wall for a commercial garage.

soil R. The floor slab will be thickened along the wall, as shown in Fig. 6-2. The slab and the foundation wall will distribute any concentrated load P. Therefore, assume that P represents a rear wheel of a loaded 10-ton truck, that the load is 3.5 tons, that the span of the floor slab is 8 ft., and that the load is distributed over a distance of 5 ft. along the shelf CD. The average load per linear foot of wall caused by P is then $(7{,}000/5) \times 5/8 = 880$ lb. The weight of the floor slab carried by the shelf may be $90 \times 4 = 360$ lb. per lin. ft. Since part of the wall is above ground and part below, its weight will be considered as that of ordinary concrete with no reduction for the displaced earth. The projections of the footing will be excluded from these calculations. The 15-in. wall at 150 p.c.f. will weigh $5.5 \times 1.25 \times 150 = 1{,}030$ lb. per lin. ft. Therefore, the approximate load per linear foot of wall at JK for design purposes is

Superstructure..........	4,100 lb.
Live load..............	880
Floor slab..............	360
Foundation............	1,030
	6,370 lb.

The width JK should be 6,370/2,500 = 2 ft. 8 in. The dimension a, in Fig. 6-2, then equals 8½ in. If t is 10 in., the tensile stress in a cantilevered plain-concrete footing may be

$$f_t = \frac{M}{S} = \frac{2{,}500 \times 0.71 \times 0.35 \times 12}{12 \times (10^2/6)} = \frac{7{,}450}{200} = 37 \text{ p.s.i.}$$

This is small, but tension in the concrete is not to be trusted in this case. Therefore, use reinforcement.

$$A_s = \frac{7{,}450}{18{,}000 \times 0.9 \times 7} = 0.066 \text{ in.}^2$$

Use ⅜-in. round hooked rods 18 in. c.c., or equivalent wire mesh.

The floor slab is to be tied to the foundation wall. The amount of steel to use is chosen arbitrarily as ½-in. round rods 24 in. c.c. They are ties, not true negative reinforcement for the floor slab; nevertheless, they will be placed near the top of the latter. These ties eliminate any danger from lateral pressures of earth and surcharge that might otherwise affect the design of the foundation. If the floor is cut loose from the wall, the latter will have to act as a small retaining wall and foundation combined, but the floor load need not be added to the vertical forces on the wall.

The longitudinal reinforcement in the wall will be computed as

$$A_s = 0.002(1.25 \times 4.67 + 0.83 \times 2.67)144 = 2.3 \text{ in.}^2$$

Five ¾-in. round rods will be used, two in the footing and three in the top as pictured in Fig. 6-2.

Special details and modifications of such a design as this will have to be made in any practical job.

6-2. Spread footings and grade beams. Most large structures have some kind of structural framework with columns which transmit heavy localized loads to the substructure. It may be that the load from the "curtain walls" of the first story and from the first floor are relatively light so that it is desirable to use large spread footings under the columns, whereas a reinforced-concrete beam between them is sufficient to support these other loads. The footings are then designed independently except for provisions for the grade beams.

This type of construction is especially adapted to industrial mill buildings in mild climates, when supported upon a stratum that lies several feet below the surface. It minimizes the excavation and the materials required to build the substructure. There may be many modifications and refinements to suit the peculiar requirements of a particular structure. One case will be illustrated by an example.

Example. Assume that Fig. 6-3(a) pictures some of the details of the construction to be used at the ground floor of a small industrial building in the Los Angeles district of California. The structure has ten 20-ft. bays, steel columns holding bracketed crane runways, 75-ft. roof trusses, and strong longitudinal roof and wall bracing. Assume that the foundation of an ordinary side-wall column, not part of a bracing bent, is subjected to the vertical loads shown in Sketch (*a*). The heavy concrete floor is to be cut loose from the foundations. The soil is fine sand capable of holding a unit load of 3 tons per ft.², but this material is from 4 ft. 6 in. to 5 ft. below the surface of the ground. The soil above it is unreliable for supporting heavy

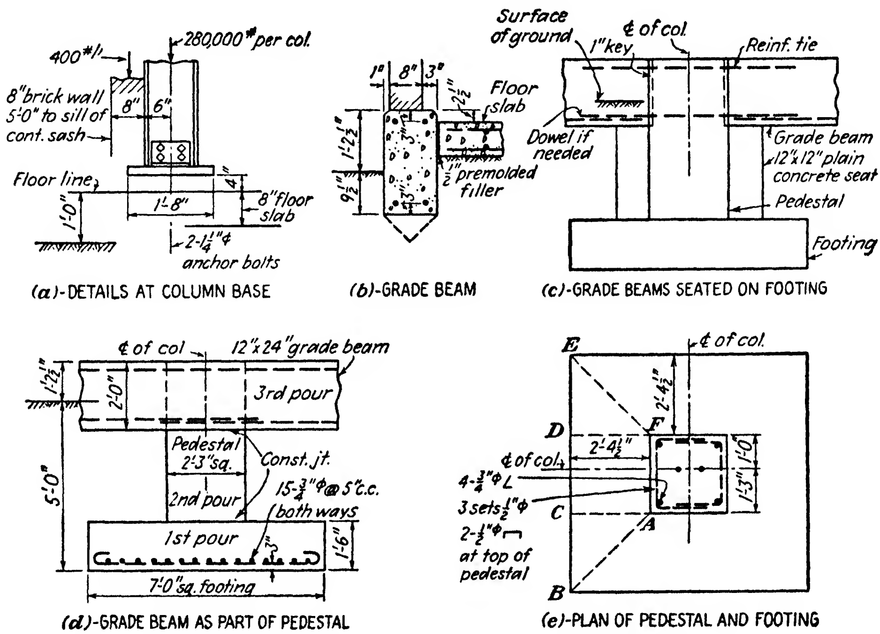


FIG. 6-3. Spread footing and grade beams.

loads. Design a typical spread footing and grade beam for this structure.

The pedestal under the steel billet should be at least 2 ft. wide in the direction of the wall. The span of the grade beam to support the brickwork will thus be approximately 18 ft. This member should be designed first. Figure 6-3(*a*) shows that the bottom of the base plate of the column is to be 4 in. above the floor line. An allowance of 1½ in. will be made for grouting under it. This elevation 2½ in. above the floor will be selected as the top of the grade beam as well as of the pedestal. The grade beam is to be under the brickwork, but a width of 8 in., equal to the thickness of the wall, seems too narrow for an 18-ft. member. Therefore, it will be made 12 in. wide as shown in (*b*). Its depth is made 2 ft. in order to extend below the ground somewhat and to be sure that it is stiff enough. The bottom of the beam might be made triangular as shown by the dotted lines. The reasons for doing so might be to reduce the bearing resistance under the beam if appreciable settlement of the main footing is expected, and to reduce frost heaving. However, the sand is not plastic, the topsoil is weak and compressible, and the efficacy of a cutting edge to resist frost action is problematical; hence, a triangular bottom is unnecessary.

A grade beam should be stiff and reinforced as a simply supported member if it is to be unaffected by any unequal settlement of the footings. On the other hand, the series of beams and footings may be tied together except at contraction joints. If so, it may be best to use a continuous beam with adequate stiffness so that it will support the masonry wall without having its deflection cause cracks in the masonry. The beam should be designed as a self-supporting member even though the earth might offer some bearing resistance under it.

The load on the grade beam in this case is dead load only. The member will be designed as a continuous beam having an end moment equal to that of a beam with fixed ends, but the center of the beam will be reinforced equally.

$$w = 400 + 1 \times 2 \times 150 = 700 \text{ lb. per lin. ft. of wall}$$

$$M = \frac{700 \times 18^2}{12} = 18{,}900 \text{ ft.-lb.}$$

$$A_s = \frac{18{,}900}{18{,}000 \times 0.9 \times 1.75} = 0.67 \text{ in.}^2$$

$$\Sigma o \text{ at the ends} = \frac{700 \times 9}{150 \times 0.9 \times 21} = 2.2 \text{ in.}^2$$

$$f_c = \frac{2 \times 18{,}900 \times 12}{12 \times 0.3 \times 0.9 \times 21^2} = 317 \text{ p.s.i. (approx)}$$

$$v_L = \frac{700 \times 9}{12 \times 0.9 \times 21} = 28 \text{ p.s.i. (approx)}$$

Therefore, use two ¾-in. round rods in the top and the bottom, as shown in Fig. 6-3(*b*). These will be extended the full length of the member in order to serve as shrinkage and temperature reinforcement also. The top rods may be lapped near the center of the span; the bottom ones, at the column. To avoid troublesome long rods during the pouring of the pedestals, horizontal dowels may be used at the columns even though the extra laps on each side cause the use of more steel.

One might well ask how the two top rods of Fig. 6-3(*b*) can be fully stressed by dead-load bending and still resist the tension caused by a drop in temperature. The bending moment will cause a small compressive stress in the lower rods. As the grade beam tries to shorten, this compression will be relieved, the upper rods will be stressed more severely in tension; then, when the strains have increased far enough, both sets of rods will be in tension. It may be that the top rods will be stressed beyond their yield point. If so, they will stretch but will not cease their resistance to bending action although the elongation may upset completely one's assumptions as to the magnitude of the negative bending moment at the column. This is one reason why the center of the beam has been reinforced for the same bending as the ends because, as the restraining moments at the ends are reduced, the positive bending moment at the center will increase. The safety factor will enable the steel at the center to hang on safely even though the beam is compelled to act as a simply supported one.

When corrugated or other flexible sidings are used, it is desirable to design the grade beams as simply supported members. It is possible to tie them together with one or two small dowels as indicated in Fig. 6-3(*c*) just to knit the structure together. If the floor is also cut loose from the grade beam, as in (*b*), the keys shown in (*c*) may be sufficient to hold the beams in line, and the dowels may be omitted. The footings may be looked upon as strong anchors that prevent the shortening of the grade beams from moving the footings, hence continuous construction is not recommended except in warm climates. Even with brick walls, the simply supported beams are preferable. This idea of intermittent anchoring is discussed more fully in the next article.

When planning the spread footing at the column, one should consider an adequate bearing for the grade beam. This may be a seat upon which the concrete of simply supported beams is poured after the footings are constructed, as shown in Fig. 6-3(*c*). For continuous construction, the beams and the tops of the pedestals may be poured monolithically, as in Sketch (*d*). The latter method is chosen here because of the assumed continuity.

The approximate total load upon this footing is the column load plus the reactions of the grade beams plus the excess weight of the footing compared to the original earth.

$$P = 280{,}000 + 2 \times 6{,}300 + 4{,}000 = 297{,}000 \text{ lb. (approx)}$$

$$\text{Area of footing} = \frac{297}{6} = 49.5 \text{ ft.}^2$$

A footing 7 ft. square will be close enough.

The outside edge of the pedestal is automatically 15 in. from the center of the column. The inside edge will be placed 12 in. from the center line in order to have the projection obstruct the floor area as little as practicable. This will cause an unimportant eccentricity of loads of about 1 in. The top of the pedestal will also be made 27 in. wide parallel to the wall.

The maximum projection of the footing beyond the pedestal is 2 ft. 4½ in. A trial *d* of 15 in. and a cover of 3 in. will be selected. Using the trapezoidal method as indicated in Fig. 6-3(*e*), the analysis of the trial footing yields the following results:

Area	Force, kips		Lever arm	Bending moment, ft.-kips
$ACDF$......	$6 \times 2.25 \times 2.38 =$	32	1.19	38
$ABC + FDE$	$6 \times 2.38 \times 2.38 =$	34	1.59	54
		66		92

$$A_s = \frac{92}{18 \times 0.9 \times 1.25} = 4.54 \text{ in.}^2$$

$$\Sigma o = \frac{66{,}000}{150 \times 0.9 \times 15} = 32.6 \text{ in.}^2$$

$$v_T = \frac{66{,}000}{27 \times 0.3 \times 15} = 544 \text{ p.s.i.}$$

$$v_L = \frac{6{,}000\left(\frac{4.75 + 7}{2}\right)1.13}{(27 + 2 \times 15) \times 0.9 \times 15} = 52 \text{ p.s.i.}$$

Therefore, the footing will be accepted with the reinforcement shown for the pedestal and footing in Fig. 6-3(d) and (e).

6-3. Walls to spread a series of equal concentrated loads. Some conditions that may make the use of grade beams undesirable and inadvisable are the following:

1. In a cold climate, the grade beam should extend below the frost line, otherwise heaving may crack it. Thus it is a large deep member automatically.

2. When reliable soils are near the surface, a grade beam will try to spread the concentrated loads to the soil even though it is not supposed to do so.

3. When the structure has concrete or masonry walls, when the soil is plastic or of unequal bearing value, and when cranes or other equipment cause large live loads, it may be best to have a strong continuous foundation under the wall to distribute the loads more evenly and minimize unequal settlements. Live loads may cause a "pumping" action on a foundation composed of separate spread footings that will tend to disintegrate the walls and the substructure.

4. The column loads may be small anyway so that shallow walls are sufficient to spread these loads, and they may be able to do so without difficulty.

5. When the floor line is 2 to 3 ft. above the ground outside, a grade beam becomes almost a wall by necessity, and it might as well be used as one.

6. When the position of the ground varies considerably along a building, it may be necessary to use walls where the height is large, and advisable to maintain the same type of construction throughout.

7. When there is a basement, some type of wall is naturally essential.

In general, unless special conditions control the design, relatively heavy column loads make it advantageous to use spread footings and grade beams, whereas small columns and heavy walls make it desirable to use continuous foundation walls. Also, large bay widths (25 ft., for example) make the first scheme preferable; short ones (like 15 ft.) favor the second.

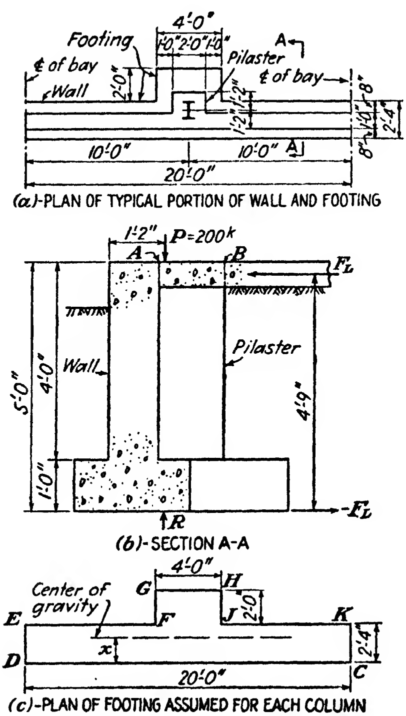


FIG. 6-4. A foundation wall with pilasters.

Because of the size and location of the columns, it is often necessary to use a pilaster on the side of a foundation wall to receive the column in order to avoid the use of a continuous wall of unreasonable thickness. It may be desirable also to have a localized spread footing under this pedestal, as pictured in Fig. 6-4(a), in order to decrease the eccentricity of the column load with respect to the center of gravity of the bearing area.

Assume the construction shown in Figs. 6-4(a) and (b). What is the maximum pressure caused by the column load alone on the soil? Sketch (c) shows the bearing area that may be assumed to support the load of a typical column if all are loaded equally. Even though the columns are not loaded simultaneously to the maximum extent, it is usually sufficient to assume that they are so loaded unless the concentrated load is very large.

Using CD as an axis, the center of gravity of the assumed bearing area may be located as follows:

Part	Area, ft.²	Lever arm	Moment, ft.³
CDEK....	$20 \times 2.33 = 46.6$	1.16	54.2
FGHJ....	$2 \times 4 = 8.$	3.33	26.6
	54.6		80.8

$$x = \frac{80.8}{54.6} = 1.48 \text{ ft.}$$

The moment of inertia of this bearing area about its centroidal axis is

$$I = \frac{20 \times 2.33^3}{12} + 46.6 \times 0.32^2 + \frac{4 \times 2^3}{12} + 8 \times 1.85^2 = 56 \text{ ft.}^4$$

The maximum pressure at GH caused by the column appears to be

$$p = \frac{P}{A} + \frac{Mc}{I} = \frac{200{,}000}{54.6} + \frac{200{,}000 \times (1.83 - 1.48) \times 2.85}{56} = 7{,}230 \text{ p.s.f.}$$

Similarly, p at CD = 100 p.s.f.

At first glance, it would seem that this eccentricity of load and the large local pressure near GH will be troublesome. This may be so if the soil is weak and if the floor is separated from the wall at A and B, Fig. 6-4(*b*). However, in practical construction, the floor may be attached to the foundation as pictured in Fig. 6-2, or it may be made so that it is in contact at A and B, Fig. 6-4(*b*). In either case, the top of the foundation cannot tilt inward, hence the overturning moment cannot affect the base seriously, and the resisting soil pressure may be assumed to be spread equally over the bearing area. If the lateral forces caused by the tendency of the walls at opposite sides of the building to tilt do not counteract each other directly, the frictional resistance of the floor slab will generally be sufficient to restrain the walls. For example, from Fig. 6-4(*b*), the lateral force F_L needed to resist tipping is

$$F_L = \frac{Pe}{h} = \frac{200{,}000 \times 0.35}{4.75} = 14{,}700 \text{ lb.}$$

A 6-in. floor slab 20 ft. wide and 20 ft. long, and a coefficient of friction of 0.5, will produce a frictional resistance of 15,000 lb. The abutting power of the earth will also help resist this overturning. Nevertheless, this tendency to tilt should not be overlooked.

In general, a foundation wall like that of Fig. 6-4 acts like a deep continuous beam that spreads most of the concentrated loads longitudinally. The weight of the superstructure's wall, or that of the first story of a curtain wall, is usually distributed sufficiently uniformly on the substructure so that its effect upon bending in the foundation wall is negligible. When the soil is firm granular material, it may be that most of the load will be resisted by the soil close to the column. However, if the wall is designed to resist the bending caused by uniform distribution, any excess load taken near the column will be on the side of safety.

Example. Assume that the wall in Fig. 6-4 is held laterally by the floor slab, that a 12-in. brick wall on the foundation averages 2,400 lb. per lin. ft., and that the trial dimensions are as shown in the illustrations. What are the added and the total pressures on the soil, and what reinforcement is needed in the substructure?

The unit pressures for bearing may be found as follows, considering that tilting is prevented:

Part		*Weight, lb.*	*Added*	*Total*
Brick wall	$20 \times 2{,}400 =$		48,000	48,000
Concrete	$4 \times 1 \times 20 =$	80		
	$2.33 \times 1 \times 20 =$	47		
	$1.33 \times 2 \times 4 =$	11		
	$2 \times 4 \times 1 =$	8		
		$146 \times 50 =$	7,300	
		$146 \times 150 =$		22,000
Earth above footing (approx)				10,000
Column			200,000	200,000
Total			255,300	280,000

$$\text{Added pressure} = \frac{255{,}300}{54.6} = 4{,}700 \text{ p.s.f.}$$

Total pressure (neglecting the floor slab and live load)

$$= \frac{280{,}000}{54.6} = 5{,}100 \text{ p.s.f.}$$

The wall will be assumed to have the uniform cross section shown in Fig. 6-5(a), neglecting the local effect of the pilaster. The upward pressure per linear foot of wall for design purposes is

$$w = \frac{200{,}000 \times 2.33}{54.6} = 8{,}600 \text{ lb.}$$

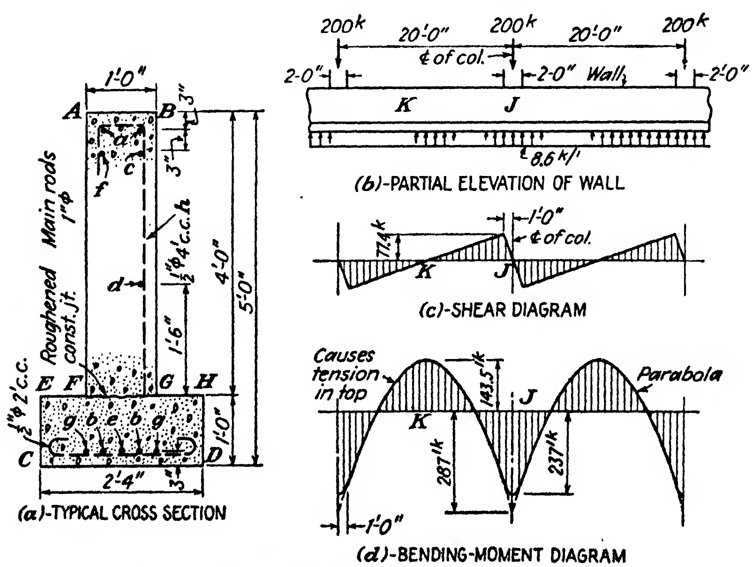


FIG. 6-5. Foundation wall supporting equal column loads.

It is assumed that the pressure under the area *FGHJ* of Fig. 6-4(*c*) is transmitted directly through the pilaster and does not affect the bending moment in the wall appreciably.

If the wall is considered to be a continuous beam with all spans loaded equally, and with the column load spread uniformly over the 2-ft. width of the pilaster, it is sufficient to assume that

$$M \text{ at } J = \frac{wL^2}{12} - \text{effect of column} = \frac{8{,}600 \times 20^2}{12} - 100{,}000 \times 0.5$$

$$= 237{,}000 \text{ ft.-lb.}$$

$$M \text{ at } K = \frac{wL^2}{24} = 143{,}500 \text{ ft.-lb.}$$

Assuming that the friction caused by pressure, the chemical bond, and the mechanical bond at *FG*, Fig. 6-5(*a*), are sufficient to make the footing and the main wall act as a unit, and that the rods are to be placed as shown in the sketch,

$$A_s \text{ at } J = \frac{237{,}000}{18{,}000 \times 0.9 \times 4.75} = 3.1 \text{ in.}^2$$

$$A_s \text{ at } K = \frac{143{,}500}{18{,}000 \times 0.9 \times 4.62} = 1.9 \text{ in.}^2$$

$$\Sigma o \text{ near } J = \frac{8{,}600 \times 9}{150 \times 0.9 \times 57} = 10 \text{ in.}^2$$

Therefore, four 1-in. round rods in the bottom and three in the top appear to be sufficient. The maximum f_c will be at the top near J. A sufficient estimate of its magnitude is

$$f_c = \frac{2 \times 237{,}000 \times 12}{12 \times 0.3 \times 0.9 \times 57^2} = 540 \text{ p.s.i.}$$

which is entirely safe.

But what should be done about reinforcement for shrinkage and temperature? If some of the rods are extended continuously through the top and bottom of the wall, they will prevent automatically any serious localized cracks due to shrinkage, even though they cannot stop that shrinkage. A large drop in temperature, however, may be a different matter. If the building is 200 ft. long, it is obvious that, with friction on the footing, bearing of earth against the sides of the pilasters and their footings, and any resistance caused by the floor slab, the ends cannot be drawn in toward the middle of the wall as a neutral point when the wall shortens. Therefore, the deformation will probably be taken up by a multitude of hair cracks. These effects will tend to add tension to that in the tensile reinforcement and relieve or eliminate compressive stresses in the concrete at and near the cracks.

The wall might be cut into units 40 to 100 ft. long by means of contraction joints as explained in Art. 6-8. The length of a unit will depend upon the importance of the case and the expected severity of temperature changes. In addition, a few more rods than required for beam action alone should be used so that there will be some reserve strength to resist the tendency to crack. The longer the unit, the more need for these extra rods. The reinforcement should be arranged so that part of the steel of the top and the bottom extends the full length of the unit, and the weakest sections should be made deliberately near the probable points of inflection because tiny cracks that occur in these vicinities will not be likely to cause much damage to the structure.

In this case, the 200-ft. length of the building is not enough to compel the use of a central contraction joint in the superstructure. If such a joint is placed in the foundation under the 12-in. brick wall, the movement of the former will tend to crack the wall in the same vicinity. Furthermore, this wall is not severely exposed. It seems desirable, therefore, to knit the entire substructure together as a unit.

The reinforcement may be proportioned to resist bending alone, then 0.002 times the cross section of the wall may be added as extra steel to make allowance for thermal effects. In this case, the added amount will be $6.33 \times 144 \times 0.002 = 1.82$ in.^{2} Three extra 1-in. rounds will be added. Therefore, rods *a* and *b* will be extended with laps, for the full length of the wall; rod *d* will be added and used similarly. Rod *c* will be added to increase the top reinforcement locally, and rod *e* will be used similarly near the column. Will this arrangement be satisfactory? No one can be sure, but it should prevent any harmful cracking. A suggested minimum area of steel for such conditions is 0.004 to 0.005 times the normal cross section of the foundation wall. If there are no pilasters or other projections, the usual 0.002 to 0.003 times the area of concrete should be sufficient. In Fig. 6-6(*a*) is shown the reinforcement for a typical "interior" bay of this wall. The footing of the wall is so deep relatively that little transverse reinforcement is needed in it. However, the footing under the pilaster is designed like part of an isolated footing, and the steel to be used in it is shown in Fig. 6-6(*b*).

The question of the amount of reinforcement to use for the prevention of cracks caused by a drop in temperature is one of those intangibles that requires the exercise of good engineering judgment. To guide the reader, the following suggestions are summarized:

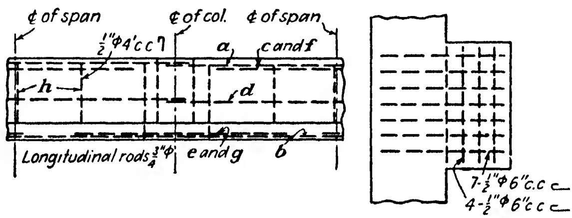
1. Determine the steel required to enable the substructure to support the assumed loads.

2. Estimate the probable range of temperature change in the particular locality. A wide daily change is more serious than a merely seasonal one.

3. Investigate the degree of exposure. Parts that are exposed to direct sunlight during the day and to cold air during the night are more endangered than are those that remain in the shade.

4. Cold climates with freezing temperatures are more dangerous than are mild climates.

5. If a small portion of the top of a wall is subjected to the effects of the atmosphere, whereas the bulk of the interior of the wall is exposed to rather uniform temperatures—as in a heated basement—the conductivity of the concrete will probably prevent serious chilling of the exposed portion. Light reinforcement will then be satisfactory.



(a)-RODS IN MAIN WALL (b)-PLAN OF RODS IN FOOTING AT COLUMN

FIG. 6-6. Arrangement of reinforcement in a foundation wall.

6. If the wall is in long sections that are anchored considerably by projections and offsets, the structure should be reinforced strongly.

7. The reinforcement should be spread over the cross section instead of being concentrated at one spot. The top and bottom of the wall are the most important locations for continuous reinforcement.

8. An important structure deserves more conservative design than does an unimportant one. Walls that are exposed to view advertise their cracks to the owner and the public.

6-4. Walls to spread a single concentrated load. When a continuous foundation wall is used in an industrial building equipped with bridge cranes, the crane load may cause a heavy concentration at any one of the columns. The wall will spread the load, somewhat as is done by a stiffening truss in a suspension bridge. If the soil is firm, the wall may not have to spread the load very far before the bearing power of the soil supports it. If the soil is weak and plastic, the wall may spread the load over considerable length, and this is probably the reason why an engineer would use such a type of construction. The stiffness of the wall naturally affects the probable longitudinal distribution of a load.

Exact analysis of the distribution of a local load by a foundation wall is probably impossible because of the many uncertainties involved, especially the unknown resistance and deformation of the soil under temporary loads However, the following assumptions and approximations are given for the guidance of the reader, and the sketches referred to are in Fig. 6-7:

1. Assume that the concentrated load P is spread uniformly over a length of wall equal to the corresponding dimension of the pilaster if used or to the width of the base plate of the column.

2. Unless the height of the wall h is less than one-fifth of the length of the bays L, assume that 75 per cent of the load P is distributed as a uniformly varying pressure over the two adjoining bays, as illustrated in Sketch (*b*), regardless of the fact that the wall may be continuous under a series of columns. This allows a 25 per cent reduction for the effect of any projecting local footings at the column points, for the fact that such a stiff wall will probably spread some of the load beyond the two adjacent bays, and for the fact that the soil near a temporary load will not have time to have the moisture squeezed out of it so as to cause appreciable compaction.

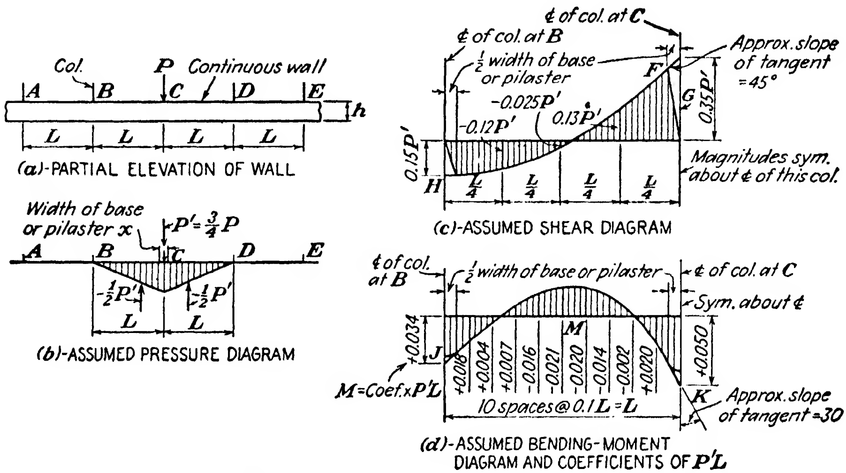


FIG. 6-7. Assumed shears and bending moments caused by a concentrated load.

3. Assume that the wall is a continuous member fixed at the adjacent columns B and D and with a constant cross section.

4. Assume that the magnitudes of the shears are as given in Sketch (*c*). Because the maximum shear at F and G is critical and because it may occur at any column, shears elsewhere need not be considered. One might ask what resists the upward reactions of the soil pressures at B and D. At least at one of the adjoining columns there will be a reaction caused by the two or more wheels of the crane trucks. The strength of the next portion of the continuous wall will also resist upward forces.

5. Assume that the magnitudes of the bending moments are as shown in Sketch (*d*). The ordinates near K and J are the critical ones for tension in the bottom of the wall; those at M, for tension in the top near the center of a bay. For purposes of plotting the shear and bending-moment diagrams, the ordinates in Sketches (*c*) and (*d*) may be used.

6. Add the shear and bending moment at any given point algebraically to those computed for the same point for any other conditions of loading that do or may exist simultaneously, and be sure that the structure can resist the combination safely.

7. Be conservative in the choice of the allowable unit stresses for bending and shear because of the probable repetition and reversal of stresses caused by the movement of the load back and forth in the structure. If v_L exceeds $0.02f'_c$, some web reinforcement is desirable. One should remember that these suggested values for shears and bending moments in a continuous foundation wall are for the purpose of giving the designer an easy way to obtain a reasonable estimate of the scale of their magnitudes rather than to have him guess their magnitudes or neglect them entirely.

At a corner of a building with continuous foundation walls, the crane load on the corner column will generally be less than those on the intermediate columns. The right-angular corner prevents the continuous beam action that has been assumed elsewhere, but the footing of the end wall will help the side wall hold up the column. In this case, it is sufficient to assume that the maximum shear in both walls near the corner column because of the temporary crane load P is $0.25P$. The bending moment causing tension in the bottom of the walls under the corner column may be called equal to zero, and the bending moment near the center of the bays caused by P and producing tension in the top of the walls may be estimated as $0.04PL$. The second column from the corner may be assumed to cause the same shear and bending as any other intermediate column.

If a continuous side wall at a corner has no intersecting end foundation wall, it is advisable to use a large footing or extension of the wall at the corner so that the substructure at this point may be designed as though the column were supported practically upon an isolated or elongated footing. The column next to the corner one may be assumed to be a typical intermediate column.

A column near a contraction joint that has no adequate keys for transmitting shear may be supported as suggested in the preceding paragraph. If the joint has proper keys for resisting shearing forces that act up at one time and down at another, the two bays of the substructure adjoining the joint may be designed for the same shears and bending moments as those suggested for the end bays at a corner having continuous side and end walls.

Special or additional reinforcement in a wall for the prevention of local large cracks caused by a drop in temperature was discussed in the preceding article. Although moving crane loads are temporary, they should be considered when one plans the steel to be added to resist this cracking. The suggested increase of 0.002 times the cross-sectional area of the wall should be made over the requirements of the combined bending moments caused by all loads.

Example. Assume that the wall pictured in Fig. 6-4 supports columns having a dead load and uniformly distributed live load of 120 kips each, and a maximum crane live load of 80 kips at any one column at a time. Design a typical portion of this wall for these conditions.

The shears and bending moments shown in Figs. 6-5(*c*) and (*d*) may be multiplied by 0.6 and later combined with those assumed to be caused by the 80-kip load. The resultant diagrams for the 120-kip load are shown in Figs. 6-8(*c*) and (*f*).

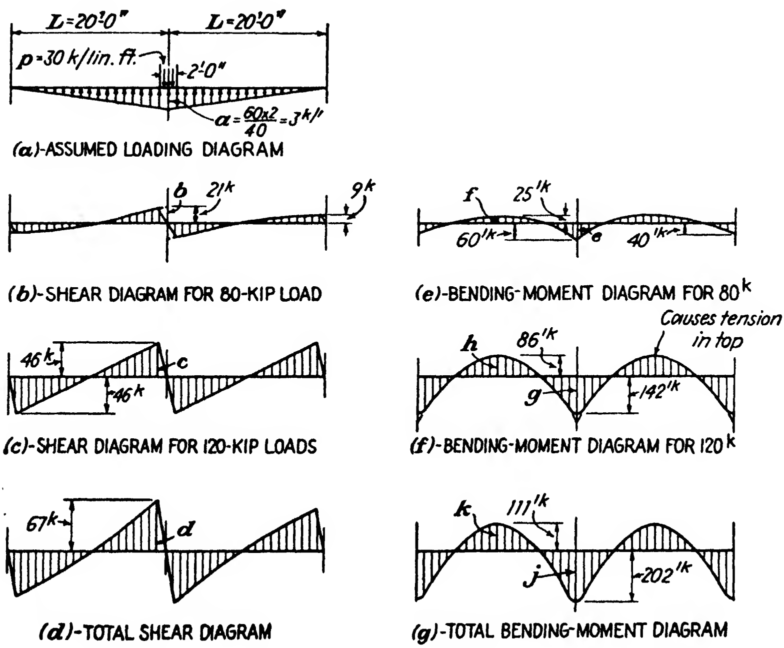


FIG. 6-8. Shear and bending-moment diagrams for a foundation wall.

The assumed uniformly varying pressure diagram for two bays is pictured in Fig. 6-8(*a*), with 75 per cent of the 80-kip load assumed to be spread by the wall in the adjacent two bays. The ordinate *b* in the shear diagram of Sketch (*b*) is obtained from Fig. 6-7(*c*), and the rest of the diagram is plotted for purposes of illustration. The maximum shear is combined with *c* of Fig. 6-8(*c*), and the combined shear diagram is pictured in (*d*). This is really the addition of ordinates at two different points, but great accuracy or refinement is not necessary because of the empirical nature of the assumptions.

Similarly, using Fig. 6-7(*d*), the diagram in Fig. 6-8(*e*) may be plotted, with the assumed net load of 60 kips. Its ordinates combined with those of the bending-moment diagram in (*f*) give the values shown in (*g*). The wall should be made strong enough to resist safely these approximated shears and bending moments, with additional steel provided to enable the structure safely to withstand temperature and shrinkage effects. The final design is shown in Fig. 6-9. The amount of reinforcement used is conservative because of the alternation and repetition of stress conditions caused by the moving load. It can be argued that the infrequent occurrence of the maximum live loads with the most severe temperature condition justifies a high allowable unit stress in the reinforcement. The author does not try to refute this argument, but the possible saving in steel is small and it is not generally worth hazarding the safety and satisfactoriness of the structure to secure it.

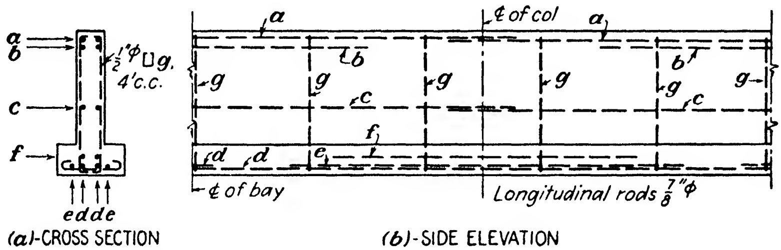


FIG. 6-9. Arrangement of reinforcement in a continuous wall.

6-5. Basement walls. When there is a basement in a building, the basement wall, if made of poured concrete, is generally so strong and stiff that, without difficulty, it will spread the column loads that rest upon it, provided the footing and the bearing power of the soil are adequate. Seldom need these walls be analyzed to see that they are safe as longitudinal beams if the longitudinal reinforcement recommended for the prevention of cracking ($0.002A$) has been used. The magnitudes of the shearing and bending stresses are very uncertain when members are so deep compared to their spans—spans between columns equal to two or three times the depth of the wall. Probably such walls cannot curve (deflect) so easily as assumed in the common theory of flexure, and shearing resistance may be unusually effective. However, there are other matters to consider.

Light columns may be supported directly upon the top of a foundation wall. This is most likely when the superstructure is made of reinforced concrete with monolithic or curtain walls between the columns. It may also occur when the foundation walls are made very thick (perhaps 24 in.) to obtain mass, rigidity, or great lateral and longitudinal strength. In most cases, the walls of the basement need not be over 10 to 15 in. thick, and then the column bases are likely to project inside the plane of the wall, as illustrated in Fig. 6-10(a). It is usually a needless waste of materials and space if the wall is thickened enough to bring its inner face to some such position as shown by line AB. It is not common practice to step back the outer face C to some such position as shown by the dotted lines near D, although this might be done.

One arrangement for the support of such a column base is shown in Fig. 6-10(b). This utilizes a pilaster at the column. The illustration also pictures a heavy floor slab that is seated on the wall, an arrangement that might occur in the case of heavy flat-slab construction for the first floor.

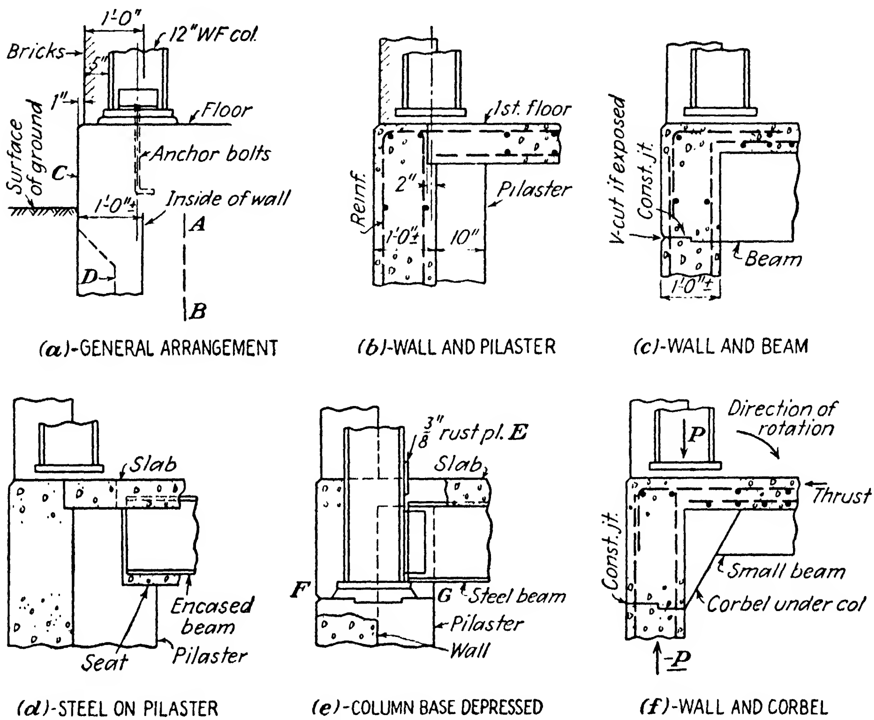


Fig. 6-10. Some details at columns on basement walls.

If there are large reinforced-concrete beams under the columns, the top of the basement wall, the top of the pilaster, and the first floor may be poured monolithically, as shown in (c) for a case where the beam is so strong that the pilaster may be omitted. If the floor is poured after the wall and pilaster are completed, the pilaster should provide a seat for the beam.

Figure 6-10(d) pictures a case in which encased steel beams are used to support the first floor. It is customary and advisable to construct the basement wall and pilasters first, then to erect all of the steelwork, and finally to build the floors. It is therefore desirable to have seats, local pockets, or shelves in the concrete construction upon which to support the steel members.

In Fig. 6-10(e), the column base is below the structural steel framing of the first floor in order to eliminate projections at or just above the floor.

This makes a neat appearance but, if the floor is likely to be wet because of operations or from washing, the steel is likely to rust at its junction with the concrete floor. Rust plates like (E) may be used to protect the main steelwork, the concrete may be sloped upward locally about ½ in. against the steel in the hope that it will shed water, or a V-shaped groove may be made at the junction of steel and concrete so that the joint may be sealed with mastic. When the columns are depressed as indicated here, it is best to have a construction joint in the wall at FG, then to pour the top of the wall monolithically with the floor rather than to leave pockets in the wall for the columns alone. The latter arrangement causes the joints to show prominently in the finished work.

If appearance and the need for usable space make it desirable to eliminate pilasters in the basement and if there is not a strong beam at the column as indicated in Fig. 6-10(c), a stepped or sloping corbel may be used as pictured in (f). This assumes that the floor will steady the wall laterally so that the wall need not act as an eccentrically loaded vertical cantilever.

There may be cases in light construction where it is desirable to support steel or wooden platforms and floors on the side of a concrete wall. Figure 6-11 shows some details that may be advantageous in such a situation in

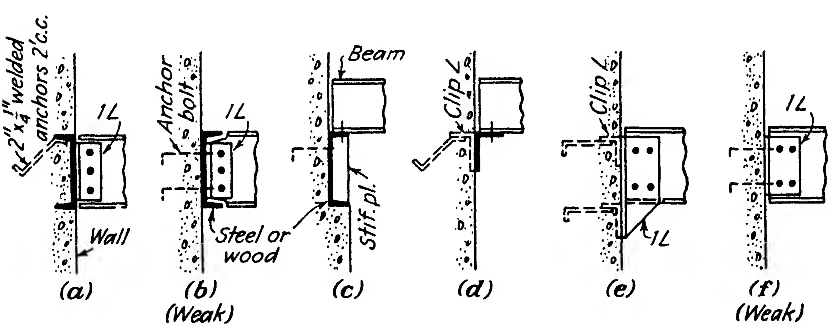


FIG. 6-11. Connections for steel framing to concrete walls.

order to avoid costly projecting shelves built of concrete, deep recesses that weaken the wall, and local pockets in which it may be difficult to erect the steelwork. One should remember that anchor bolts embedded in concrete, as in (b) and (f), are no stronger in resisting shear than the bearing value of the concrete against the outer 1 or 2 in. of the embedded bolt. The embedded structural pieces shown enable the bolts to bear against steel and the concrete against a part having considerable bearing area. These structural parts should be so anchored that they will not rip out because of eccentric loading and tensile forces.

The lateral pressure of the soil outside of a high basement wall may cause considerable bending in the latter as a vertical slab. The wall will be supported at its top and bottom ordinarily, as pictured in Fig. 6-12(a).

The wall should ordinarily be designed as a simply supported member with a loading diagram similar to that shown in this sketch. The bending-moment diagram may be assumed to be as pictured in Sketch (*b*). The vertical load will relieve a little of the computed tension in the vertical reinforcement, but this may be neglected because its effect is small compared to the uncertainties in the assumption of loads. End restraint of basement

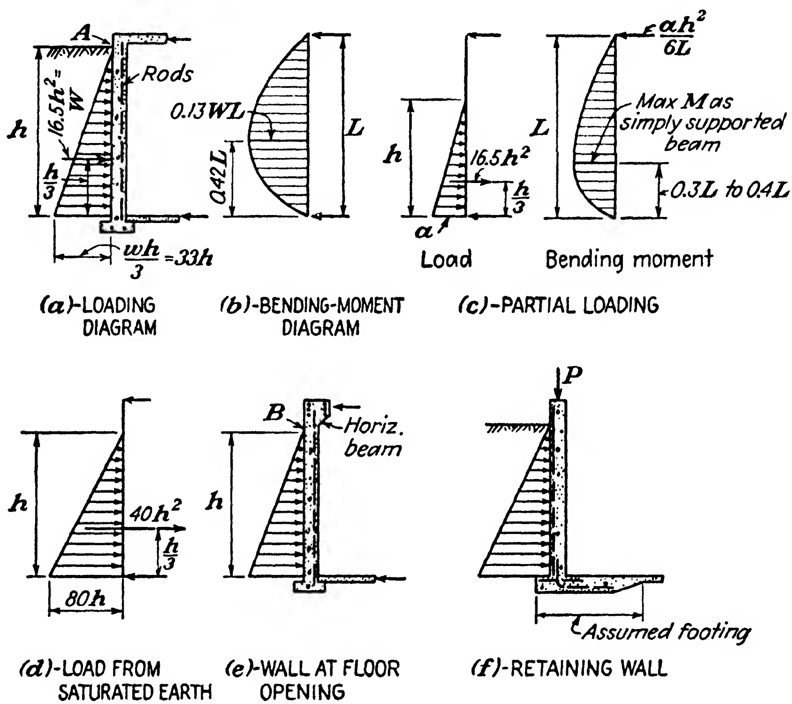


FIG. 6-12. Pressure diagrams and bending moments assumed to be applicable to a one-foot vertical slice of a basement wall.

walls may be provided in some cases, but its assumed effect upon the economy of reinforcement is likely to be deceptive. When a large part of the height of a basement wall is above ground, the lateral bending moment will be small and possibly negligible, or it may be assumed to be as shown in (*c*). When the basement is two stories deep, it is probably desirable to design the wall as a two-span continuous vertical slab with a uniformly varying lateral load.

If water pressure is likely to be applied against the basement wall in addition to the soil pressure, the intensity is not likely to be equal to hydrostatic pressure plus earth pressure acting separately. It is generally sufficient to assume that the two cause a pressure like that shown in Fig.

6-12(d). If the watertable is very low, the hydrostatic pressure may be neglected; if it is within 3 or 4 ft. of the surface of the ground, it should be assumed at the surface. The effectiveness of drainage is a matter to be estimated by the engineer for each special case.

When part of a basement wall is opposite an opening in the first floor, the top of the wall may be reinforced as a beam to span horizontally across the opening. If the open space and the loads are too great, the top may be enlarged as shown in Fig. 6-12(e), or the wall may be designed as an L-shaped retaining wall somewhat as indicated in (f). In the latter case, the vertical load of the superstructure should be included when the stability of the assumed effective unit is tested and when the intensity of pressure upon the ground is estimated.

Pipes and ducts for utilities generally must pass through basement walls and probably through other foundation walls, too. It is frequently desirable to provide large rectangular holes through the walls when they are poured so that the piping can be installed later, thus avoiding delay and minimizing errors of location of pipes. The spaces around pipes can then be packed with concrete or mortar after all connections are made.

The details to be used at the bottoms of basement walls may be affected by special local conditions. Figure 6-13 shows some construction that may be suitable for many cases. Heavy basement floors to resist uplift are discussed in Chap. 7.

6-6. Waterproofing basements. Admixtures in the concrete of basement walls may be beneficial in preventing leakage if they increase the density or reduce the porosity of the concrete. However, they do not stop leakage at construction joints and cracks. It is obvious that the joints in a basement wall built of precast blocks are likely to be planes where leakage will occur easily. Coatings applied to the inner side of a basement wall that is found to leak *may* be helpful, but this remedy is a last resort and of dubious efficacy. It is best to prevent leakage by proper planning in the original design.

Drainage lines may be placed outside of basement walls as indicated in Figs. 6-13(c) and 6-15(a). To be effective, these pipe lines should have open joints that will let in the water but will not become clogged, and will not permit the pipe line to be closed by silt. The pipes should be of vitrified clay, cast iron, or other noncorrodible material. Uncalked bell-and-spigot joints, wrapping of joints with tar paper, embedment of the pipes in coarse screened gravel or crushed stone, and the insertion of burlap, tarpaper sheets, salt hay, and similar materials between the stone and the earth backfill are all effective means for securing a minimum of silting. Furthermore, the drainage lines should discharge at an outlet which will let the water flow away, which will not become submerged, and which will not be closed by freezing. It is helpful if the lines can be cleaned or flushed out by having suitable risers and Y-connections.

Incidentally, drains placed in and under a basement floor to prevent ponding should have closed joints, and the operation of their discharge

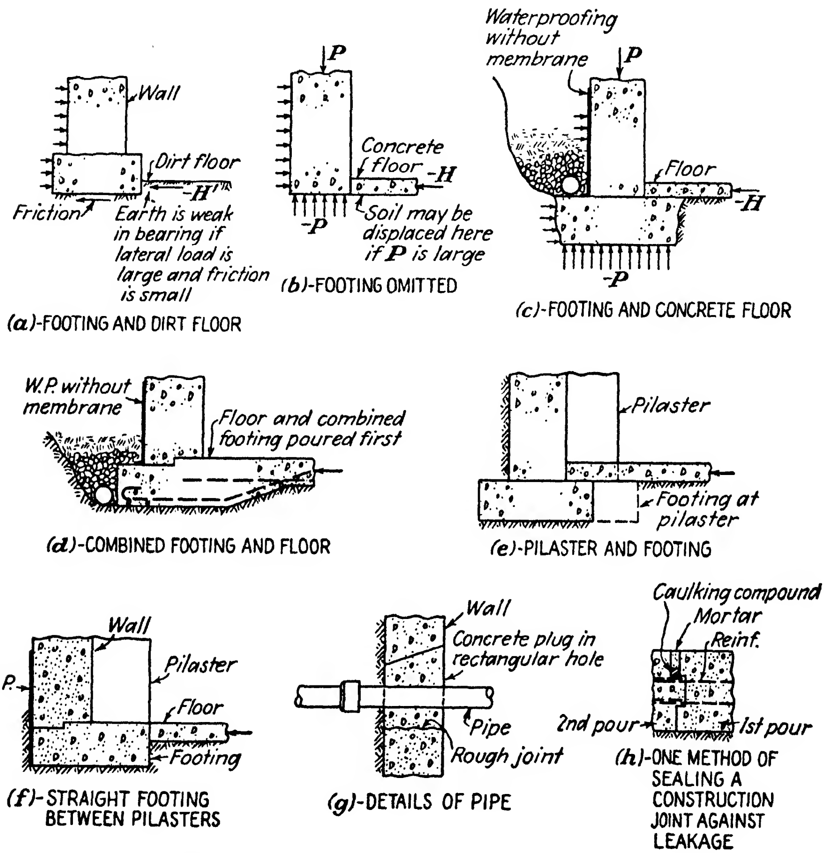


FIG. 6-13. Some details at bottom of basement wall.

facilities should be reliable. Too frequently, such installations have been made without regard to these matters and to the elevation of the ground water. The result has been the flooding of the basement and the realization that water may flow either way in a pipe.

A bituminous emulsion or mastic coating without a membrane may be used upon the outer surface of a wall as shown in Figs. 6-13(*c*) and (*d*). This means that space must be available for its application. Such a coating is very beneficial, but it cannot be depended upon to resist leakage permanently when subjected to hydrostatic pressure that acts for at least a day or two at a time. It is desirable to have such coatings built up by two successive moppings.

When it is important to prevent the penetration of dampness and actual leakage of water through a basement wall, the basement should be designed and built so that the water can be stopped at the outside surface of the structure and so that the basement walls and floor can resist any resultant hydrostatic pressure. One of the most effective means of waterproofing a basement is the use of a continuous bituminous membrane forming a sort of waterproof pan that fits under and around the basement. In this membrane, tar or asphalt is generally used to secure the waterproofing properties, whereas fabric is included to provide toughness and reinforcement.

The general nature of a membrane waterproofing is illustrated in Fig. 6-14. Much of the quality of the product depends upon the skill with which the

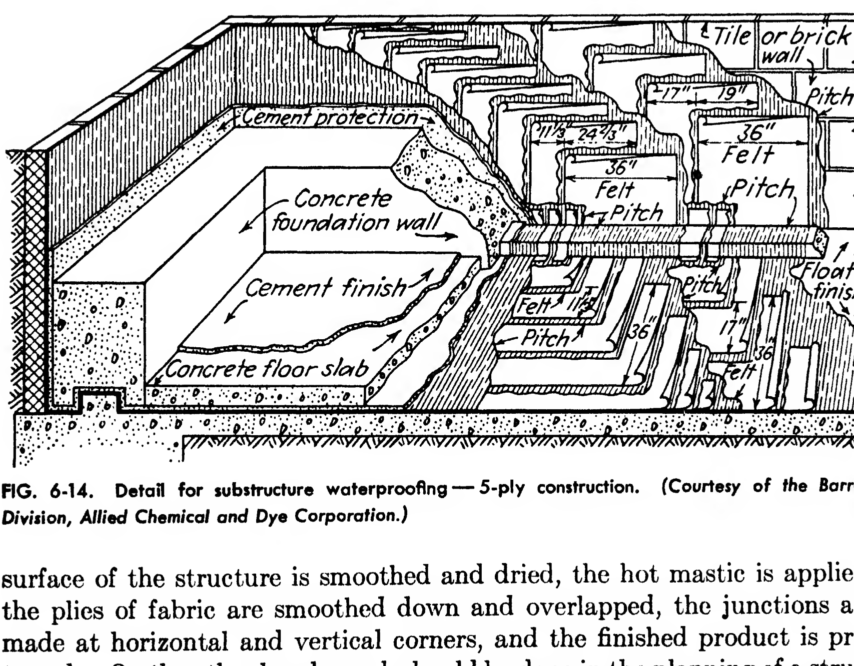


FIG. 6-14. Detail for substructure waterproofing—5-ply construction. (Courtesy of the Barrett Division, Allied Chemical and Dye Corporation.)

surface of the structure is smoothed and dried, the hot mastic is applied, the plies of fabric are smoothed down and overlapped, the junctions are made at horizontal and vertical corners, and the finished product is protected. On the other hand, much should be done in the planning of a structure to make it possible and easy for the contractor to secure good results.

Some suggestions are given below, and the sketches referred to are in Fig. 6-15:

1. When conditions are not sufficiently serious to cause waterproofing of the basement floor, the membrane used on the side wall should seal the construction joint at the top of the footing, as indicated at A in Sketch (a).

2. Adequate clearance should be provided between the concrete wall and the side of the excavation, as shown in (a). Otherwise, the workmanship

may be poor, as one can learn for himself if he tries to apply hot asphalt and fabric in a restricted space.

3. If the membrane is not protected, backfilling and other operations may damage it. Old bricks laid on edge in lean mortar as pictured in (*a*) are

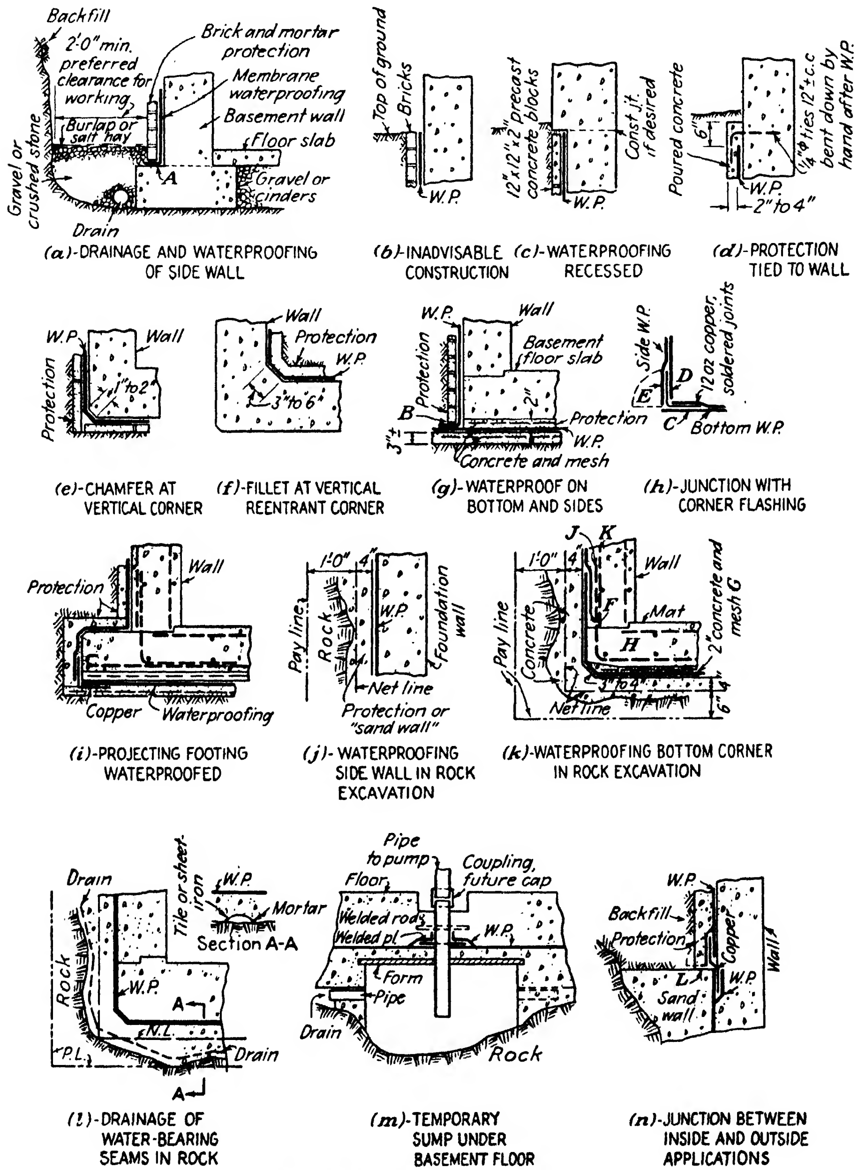


FIG. 6-15. **Some details of construction for membrane waterproofing of basements and other underground structures.**

suitable for this, and so are 2-in. precast concrete blocks. A poured-concrete covering 2 in. to 4 in. thick is even better, but it is more costly. Sheets of plywood and pieces of old lumber may offer temporary protection for the membrane, but their use in important construction should be questioned.

4. The top of such a protective coating should not be simply brought to the top of the waterproofing and then left there, as in (*b*), because this may fail to help prevent the eventual penetration of water behind the top of the membrane. Some details for better construction are shown in Sketches (*c*) and (*d*).

5. Projecting corners of the substructure should be chamferred as shown in (*e*) so that the membrane can be built without the danger of tearing and serious wrinkling of fabric.

6. Reentrant vertical corners should be widely filleted as pictured in (*f*) so as to avoid air pockets, poor overlaps of fabric, and harmful wrinkles.

7. When the basement floor is to be waterproofed, the membrane under the structural slab should be laid upon a thin concrete base, as pictured in (*g*). The membrane on the wall may be lapped over the horizontal layer later on, as indicated at *B*. However, this junction may not be effective in resisting water under pressure. It is likely to be safer if soldered copper flashing is lapped into the bottom membrane as shown diagrammatically in (*h*); then the flashing may be bent up, folded (or possibly soldered) at the corners of the building, and sealed into extra plies used at the bottom of the vertical membrane. It is apparent that heavy pressures applied by the structure may cause the mastic to squeeze out from under the walls. Therefore, wide copper sheets soldered at the joints may be used as flashing, but the copper should be thin (8 oz. per ft.^{2}), otherwise it may be too stiff to lie flat and in complete contact with the bottom. On the other hand, a footing may be used as in (*i*) to reduce the unit pressure on the membrane. This trouble may also apply in the case of interior columns that carry very heavy loads. Another feature to notice in Sketch (*g*) is the 2-in. protection of concrete placed over the waterproofing so that the erection of reinforcement, the pouring of concrete, and other operations will not damage the membrane.

8. When the basement is in rock excavation, a concrete "sand wall" should be poured against the rock, as indicated in Sketch (*j*), so that the membrane waterproofing can be applied to a smooth dry even surface before the main structure is built. This minimizes the amount of rock excavation by the elimination of outside working space. Of course, the rock will be irregular, and the concrete should not be expected to stick if it is merely plastered upon the rock; at least 3 or 4 in. should be provided between the waterproofing and the *net line*—the line within which the rock must not project—in order to make sure that the sand wall will be self-supporting.

It is often necessary to build the sand wall in successive layers 5 or 6 ft. high because of the cost of forms and the difficulty of placing concrete in such thin sections.

9. When the bottom as well as the sides of a basement in rock excavation must be waterproofed, the construction may be made as shown in Sketch (*k*). The bottom membrane is extended up the sides at *F* after the rock is lined with concrete. The protecting concrete cover *G* is placed, and then the waterproofing is applied on the side walls. If the floor or mat *H* is poured before the membrane is applied on the sides, the rods *J* projecting up to bond with the wall reinforcement *K* to reinforce the corner will seriously interfere with the construction of the waterproofing.

10. When water-bearing seams are in the rock, hydrostatic pressure is likely to push off the sand wall. One way to prevent this is to install split-tile or sheet-iron drains over the seams, as indicated in Sketch (*l*). These drains may then be connected to a temporary sump and pumps, as indicated in (*m*); later the sump may be sealed. In such a case, it is probable that the sand wall should not be constructed more than a few feet high before the membrane and the pressure-resisting main structure are built inside it, and within 3 or 4 ft. of its temporary top.

11. Generally there will be a position near the top of the rock excavation at which it is possible and desirable to change from inside waterproofing to normal outside operations, as shown in Sketch (*n*). Copper flashing in the lower membrane may be bent outward at *L*, then it may be bent up again and joined into the upper membrane. This work is difficult, and its results may not be reliable. It may be advisable, in an important case, to extend the sand walls upward as a 6-in. reinforced-concrete box, then to apply the waterproofing on the inside only.

12. Pasting waterproofing overhead on ceilings and surfaces that slope forward (as when undercut) is almost impossible and should be avoided. It is often advisable to increase an excavation so that shelves and slopes can be waterproofed from the top and outside. Of course, the top of a conveyor tunnel or compartment in the bottom of an open excavation can have its waterproofing joined to the side-wall membranes by simply lapping it over the latter, or making the junction by means of the principles illustrated in (*n*). The membrane on the top of the structure should be protected by a layer of concrete. It is preferable to have mesh reinforcement in this layer, and to extend the top protection over that of the sides.

A membrane should be composed of at least two plies of fabric if it is to be reliably waterproof. Three or four plies may be desirable when the hydrostatic pressures are large and when some ductility of the membrane is needed because of possible strains and small motions of the structure. A four-ply membrane will be approximately 3/8 in. thick.

When piping must pass through a membrane waterproofing, special details are needed at the junction. Pasting of the membrane around the pipe may be sufficient when there is little hydrostatic pressure. In serious cases, welded plate flanges may be used to lap under or into the membrane as indicated in Fig. 6-15(m). The rod welded to the pipe in this illustration is to prevent rotation of the pipe when connections are made.

Bricks laid in and coated with mastic have been used as waterproofing. Such construction is difficult to build, the numerous joints add to the uncertainty of its tightness, and settlement of the bricks because of the plastic material in the joints may rupture the waterproofing near its top.

When dampness occurs in a basement because of condensation (sweating), membrane waterproofing will not stop this action. Proper ventilation may remove the trouble except when warm, moist air is brought in contact with cool walls, whereupon the trouble may be aggravated. Slight heating of the basement will cause the desired evaporation in most cases.

6-7. Miscellaneous details. There are various details of long foundation walls that should be planned in accordance with the requirements of practicable engineering and construction. One of these is the location and type of joints to be used in a wall.

The quantity of concrete that can be poured in a limited time or as one continuous operation may be inadequate for the construction of the wall as a complete unit. Economy and reuse of forms make it desirable to build the wall in relatively short lengths, or in short lifts of 6 to 10 ft. The permissible construction joints should be located and shown on the plans by the engineer instead of leaving such matters to be determined by anyone who happens to think of them.

Horizontal construction joints may not be troublesome unless they are located at points where large shearing forces exist. Even then the friction caused by the vertical pressure, and the bond of the concrete to the previous pour, may be sufficient without the use of special keyways. In Figs. 16-13 and 16-15 are shown some arrangements for construction joints between footings, walls, and floors; and they indicate that a basement floor may be used as a strut between opposite walls. However, construction joints may be influential in their effects upon the planning of the reinforcement.

It is desirable to avoid the necessity of supporting long vertical rods while a little of their lower ends is covered with concrete; they are also likely to interfere with further formwork. However, a rod may ordinarily be used through two pours if the projection above the top of the lower pour is not over 150 to 200 times the diameter of the rod; greater lengths may need lateral support. When vertical rods are spliced, it is convenient to arrange the reinforcement as suggested in Fig. 6-16(a). This joint is assumed to be at a point where the wall is not laterally supported. At points A and B the edges of the first pour may be troweled for a depth of 1 in. when a neat straight joint is desired, or a V-cut may be used at these points to separate the pours so that any difference in color and texture of concrete will not be too apparent. Figure 6-16(b) shows an arrangement

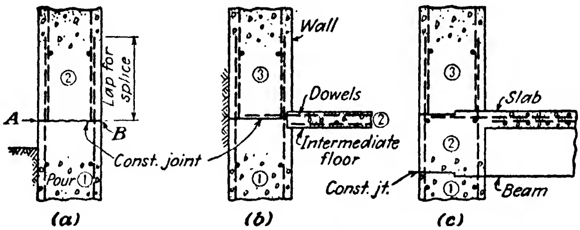


FIG. 6-16. Some details at intermediate horizontal construction joints in heavy concrete foundation walls.

for construction joints that may be used when a light intermediate floor or platform is to be inside the wall. Sketch (c) pictures construction that may apply when the intermediate floor is heavy.

Vertical construction joints are likely to endanger the strength of a foundation wall. It is obvious that such a joint located as shown in Fig. 6-17(a)

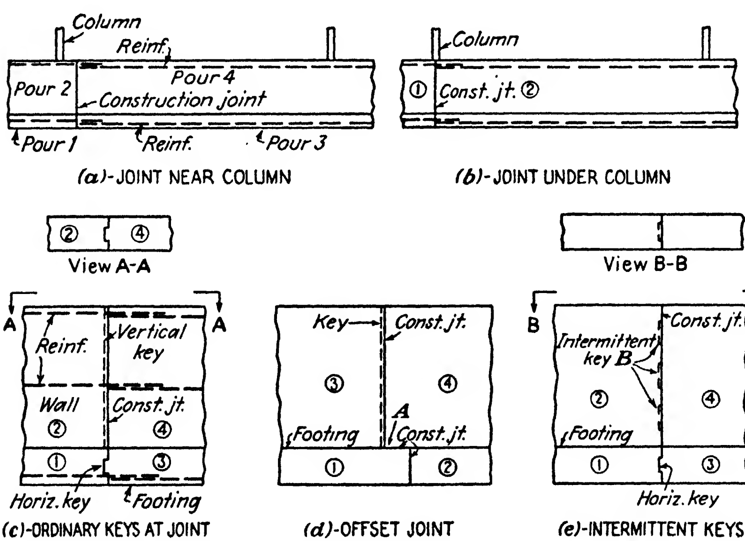


FIG. 6-17. Vertical construction joints in a continuous foundation wall.

constitutes a plane of weakness that endangers the spreading of the load from the adjacent column because the reinforcement should not be relied upon as dowels to serve this purpose. A joint located under the center of a column, as in (b), may be safe, but this is often inadvisable. It is preferable to have a construction joint at the center of the space between columns, or at other points of small shearing stress.

Keys may be used at vertical construction joints, as shown in Fig. 6-17(*c*). The vertical key will not be effective in resisting vertical shearing forces, and the key in the footing alone may be too weak for this purpose. The scheme shown in Sketch (*d*) may be useful if the shearing force is always downward at the right, but it is weak when the reverse occurs. The intermittent keys shown by *B* in Sketch (*c*) are useful in resisting both horizontal and vertical shearing forces. They may be made easily by attaching short pieces of boards or planks to the inside of the end form of the first pour.

Contraction joints (and so-called *expansion* joints) in heavy foundation walls are generally troublesome. It is often difficult to determine whether to cut the foundation into units of reasonable length or to endeavor to knit it together as an entity. If the superstructure is sufficiently flexible and if the walls are composed of units that permit slight adjustment, the foundation walls should be cut by contraction joints into units from 60 to 100 ft. long, with joints in the floor slab to match those in the walls, although the floor should have many more such joints. If there is a basement, and concrete floors supported by the basement walls, it is preferable to try to make the substructure act as a unit unless its length exceeds 200 or 300 ft. In such a case, it is possible to minimize the harmful effects of shrinkage by pouring the walls in alternate units, then filling in the intervening sections 1 to 3 weeks later. It is probable that basement walls that are not largely exposed to the weather will not be subjected to a wide range of temperature change because the air inside remains relatively uniform in temperature. It is apparent that building foundations are not affected so violently by changes in weather as are retaining walls and similar long exposed structures.

If contraction joints are to be used, they should be located at proper points. A few suggestions regarding their locations are the following:

1. They should be at points having small shearing action. A location similar to that of the joint in Fig. 6-18(*a*) is inadvisable because of the interference with proper distribution of the column load.

2. When a double column is used at a contraction joint in the superstructure, the joint in the substructure should be in the same plane, as indicated in (*b*).

3. If the wall has offsets and corners, as in (*c*), the joints should be at such positions as *A*, *B*, and *C* so that the substructure is divided into parts that permit small motions to occur without cracking the wall. Corners and offsets may act like anchors or as points of weakness. The longitudinal motions should open the joints, not shear them as at *D*.

4. Keys should be used when vertical or horizontal shearing forces are to be resisted at the joint. They should be strong but not too deep, and they

should have beveled edges. The end of the first section of wall and the keyways should be painted with asphalt, paraffin, oil, or some other material to prevent bonding the two sections together. A sample detail is shown in Sketch (*d*).

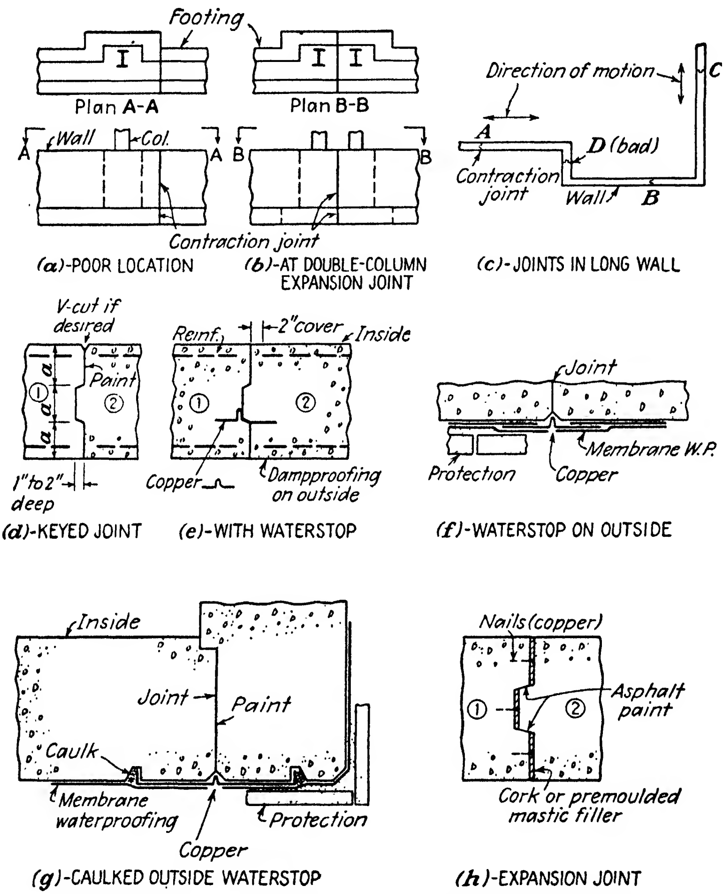


FIG. 6-18. Some details of contraction joints in foundation walls.

5. Contraction (and construction) joints may leak. One type of waterstop used to prevent this is pictured in Sketch (*e*). The continuous soldered copper strip should be stiff enough to hold its shape during the placing of the concrete. Such a strip placed in a horizontal construction joint is likely to be bent down seriously by construction operations. Some other types of waterstop used with membrane waterproofing are shown in (*f*) and (*g*).

6. If a compressible filler is to be used in an expansion joint, as shown in Sketch (*h*), the filler should be fastened mechanically so that it will remain in place initially and cannot fall out later.

7. Of course, reinforcement should not pass through a contraction joint.

When planning the reinforcement of walls that are subjected to longitudinal and lateral forces, the designer should be sure that the structure is tied together properly. The following comments refer to the correspondingly lettered portions of Fig. 6-19:

(*a*) Hooks on reinforcing rods are useful under conditions that do not afford adequate development of bond otherwise. However, when used as at *A*, they are harmful because of the concentration of stress at one place.

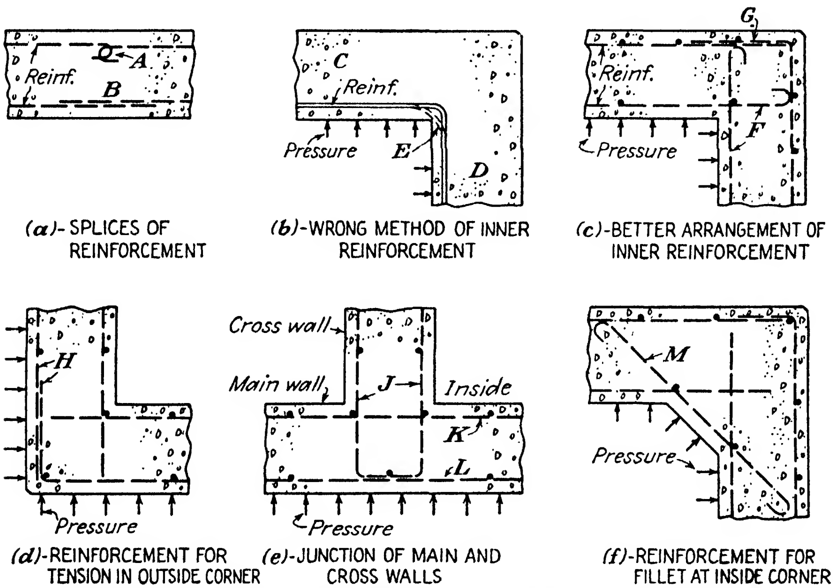


FIG. 6-19. Some details of reinforcement for heavy basement walls.

It is better to splice the rods by unfolding the hooks and so secure a simple lap of adequate length, as at *B*.

(*b*) Reentrant corners that are subjected to forces that try to separate wall *C* from *D*, or to increase the angle between them, tend to crack. Tension in rod *E* causes it to try to straighten out as shown by the dotted line, thereby producing a tendency to spall off the concrete and permit large cracks before appreciable resistance is offered.

(*c*) Reentrant corners may be reinforced better if made as shown here. Rods *F* reinforce the inner corner. The lap at *G* is desirable but is not required to resist bending.

(*d*) When tension exists around a projecting corner, rods *H* should be lapped.

(*e*) At such an intersection as this, rods *J* should be bent as shown or form a U; hooks are less desirable. Rods *K* and *L* should go straight through or be lapped.

(*f*) Large fillets in a reentrant corner are beneficial when subjected to compression. If the corner is in tension, they avail little unless rods *M* are used.

Foundations for wooden floors or superstructures should be planned so as to prevent the access of termites to the woodwork. Suggested construction is shown in Fig. 6-20, and the following suggestions are given:

1. Copper termite guards should be placed beneath the woodwork and on all pipes, piers, and parts that may serve as a means of access for the insects.

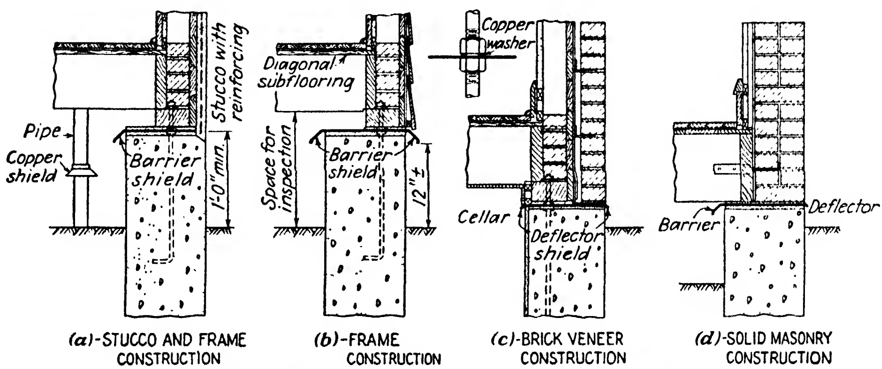


FIG. 6-20. Some construction used to prevent access of termites to timbers of a building. (Courtesy of American Brass Co.)

2. All guards and woodwork should be as high above the ground as it is practicable to have them, 1 to 2 ft.

3. Enclosed spaces without basements should be accessible for inspection, and a clearance of 2 to 2½ ft. is none too large for one to crawl around to examine the structure. If inspection is not easy, it will seldom be done.

4. Cracks in foundation walls may become tunnels for the termites. It is obvious, therefore, that a hollow-block foundation wall is especially hazardous unless the voids are filled with concrete or the upper part of the wall is made of solid brickwork or concrete.

5. Poison of the right kind placed in the ground alongside a foundation wall will be helpful in protecting the structure for 1 to 5 years, but this is a temporary expedient.

PROBLEMS

For all the following problems, assume that the cover over the reinforcement is 3 in. and that the minimum frost depth is 4 ft.

6-1. A basement wall is 10 in. thick and supports a load of 4 kips per lin. ft. of wall. The soil is silty clay. Design a footing for this wall.

6-2. The basement wall under a small apartment house is somewhat similar to Fig. 6-1(*d*). The wall is 14 in. thick and 10 ft. high. The load of the brickwork, etc., is 6 kips per lin. ft. of wall. The allowable bearing on the soil is 1½ tons per ft.^{2} Design a footing for this wall.

6-3. A factory is to have a foundation composed of spread footings and grade beams like Fig. 6-3(*d*). The columns are 25 ft. c.c. and support loads of 300 kips each. The pedestals are 2 ft. 6 in. square. The brick walls weigh 1.5 kips per lin. ft. of wall. The grade beams are 14 in. wide and 30 in. deep with three 1-in. round rods in the top near the column and in the bottom near the center but only two in the top and bottom elsewhere. The footing is 8 ft. square, 21 in. thick, and reinforced with two bands of seventeen ¾-in. round rods 5 in. c.c. The top of the concrete is 1 ft. above ground; the bottom, 4 ft. 6 in. below grade. Is this satisfactory?

6-4. Design grade beams and footings for a factory wall having concrete columns 18 in. wide, 24 in. deep, 22 ft. 6 in. c.c. The column loads are 350 kips each; the wall load, 2 kips per lin. ft. The floor is 6 in. above ground. The allowable bearing value of the earth is 3 tons per ft.^{2}

6-5. A foundation wall is to be of the type shown in Fig. 6-4. The columns are 22 ft. c.c. and support loads of 240 kips each. The wall load from the superstructure is 1.2 kips per lin. ft. The floor is 1 ft. above ground. Column bases are 16 $\times$ 24 in. The allowable bearing value of the soil is 4 k.s.f. Design the foundation.

6-6. Assume a foundation exactly like Fig. 6-4 except that the wall footing projects 10 in. on each side of the wall and the column loads are 275 kips. The footing reinforcement is ⅜-in. rounds 12 in. c.c. for the wall and ½-in. rounds 6 in. c.c. under the pilaster. The wall reinforcement is as follows:

Two ⅞-in. rounds, full length top and bottom

One extra ⅞-in. round, 14 ft. long in top for central portion of bay

Two extra ⅞-in. rounds, 12 ft. long in bottom at the column

Is this construction satisfactory?

6-7. A row of crane columns in the center of a factory is to be founded upon a continuous wall. The columns are 20 ft. c.c., have dead loads of 100 kips each, and are to be on base plates 16 $\times$ 36 in. Assume the crane loads to be equivalent to a concentrated force of 300 kips on any one column, or 150 kips on two adjacent columns. There is no wall in the superstructure between the columns. The ground has a safe bearing capacity of 4 k.s.f. Design the foundation.

6-8. A reinforced-concrete warehouse has exterior columns 21 ft. c.c. The construction at the first floor is similar to that of Fig. 6-10(*b*) except that the columns are 20 $\times$ 24 in. Their outer faces are covered with one layer of bricks so that they are 6 in. inside of the outer surface of the concrete wall. The column loads are 400 kips each; the wall and floor loads average 4 kips per lin. ft. The distance from first floor to basement floor is 11 ft. The first floor is approximately 4 ft. above the ground, and a small window is in each bay of the wall. The soil is medium sand and is well drained. Design the basement wall.

7

MATS

7-1. Introduction. The word *mat* is used to denote a substructure that transmits its loads to the soil by means of a continuous slab that covers the entire area of the bottom of a structure, like a floor. The use of a mat may be looked upon as the result of a situation in which the low bearing value of the soil would result in such large isolated footings that it is logical to join them all together and transmit the loads to the entire area directly under the superstructure.

There are several variations of this basic construction, a few of which will be discussed as though they were separate structures. In engineering practice, there may be the need for combining and modifying the general features presented in order to meet the exigencies of particular problems. The engineer's first step in any case is to plan the substructure so that it can be built practicably and economically, and so that he thinks it will support the loads safely. The tentatively chosen dimensions and members are to be tested by analysis, and then revised if necessary. A preliminary analysis is used to obtain guiding data regarding the unit working stresses in the materials, and the thicknesses and reinforcement needed. However, the methods of analysis and the assumptions should be such that the designer is reasonably certain that they will enable him to obtain a structure that will serve his purpose safely.

Here again great refinement of calculation is not always justified or practicable because of the uncertainties of the action of soils and of such short thick members that are often arranged in complicated and multiple systems. The assumed supporting systems and the suggested methods of preliminary analysis shown herein are believed to be practicable, to be easily understood, and to yield safe results. The illustrations are purposely simplified, and the general scheme of reinforcement is merely indicated. Other procedures utilizing greater refinement of calculation may be used as a final check of the preliminary design, recognizing the mat as a highly indeterminate structure.

7-2. Uniform mat. A type of mat that may be constructed easily is one of uniform thickness supporting individual columns, or columns that are on individual pedestals, as shown in Fig. 7-1. Assume that this illustration pictures a proposed substructure to be used as the basement of an apartment house founded upon plastic soil, such as soft clay. The fact that a basement floor is needed anyway is a secondary reason why one might wish to combine this with the structural supporting system in the form of a continuous mat. The illustrations are simplified by the omission of waterproofing because the latter should be outside of the structural mat, and the waterproofing and its appurtenances would contribute little or nothing to the support of the superstructure.

Figure 7-1(*a*) shows an assumed vertical section through this basement. The outside walls have pilasters at the columns, but the mat itself is to serve as the footing, and the outer walls can distribute the column loads P_1 along the edge of the mat. The interior columns are assumed to go down to the pedestals shown projecting above a portion of the basement floor. These pedestals serve the same purpose as the capitals and drop panels of a conventional flat-slab floor; *i.e.*, they spread the critical shearing and bending sections over enough perimeter to prevent harmful concentrations in local spots. If the columns are used without any enlargement of the base, the column is likely to punch through the slab unless the latter is very thick. The tension in the bottom of the mat under the column is also likely to be excessive. A flared base on the column, like an inverted capital, would serve to spread the load even better than a pedestal of the same area of bearing on the mat.

Such a mat as this in Fig. 7-1(*a*) should be thick, stiff, and strong. It will generally be loaded with an upward pressure of 500 to 1,500 p.s.f., or perhaps more. It seems, therefore, that it is not practicable to analyze it strictly in accordance with the empirical codes often used for the analysis of flat-slab floors; *e.g.*, the 1940 Report of the Joint Committee. Those floors are thin, relatively flexible, and designed for much lighter loads.

It is reasonable to assume that the mat is so stiff and the load so constant that plastic soil will compress and adjust itself so that each column load will spread almost uniformly under the mat in the general vicinity of that particular column. For example, the total unit pressure under the rectangular area *DEFG*, Fig. 7-1(*b*), may be assumed equal to one-fourth of the total loads on the columns at *D*, *E*, *F*, and *G* divided by the area of *DEFG*, plus the weight of the mat per square foot. As the last item does not affect the shears and bending moments in the mat, the net load caused by the columns alone will be considered when analyzing the mat except for the portion close to the walls, where the weight of the basement walls is included in the assumed loads P_1, P_5, etc. For the purpose of computing assumed average pressures under the slab, near the walls, the outer column loads will be treated as though they were concentrated at the pilasters. Therefore, the unit pressure for use in the design of the mat is

$$p \text{ for } DEFG = \frac{\frac{1}{4}(2P_2 + 2P_3)}{A} = \frac{\frac{1}{2}(500 + 550)}{20 \times 22} = 1.19 \text{ k.s.f.}$$

$$p \text{ for } DETS = \frac{\frac{1}{4}(P_2 + P_3) + \frac{1}{2}(P_5 + P_6)}{A}$$

$$= \frac{\frac{1}{4}(500 + 550) + \frac{1}{2}(240 + 260)}{(1 + 20) \times 22} = 1.11 \text{ k.s.f.}$$

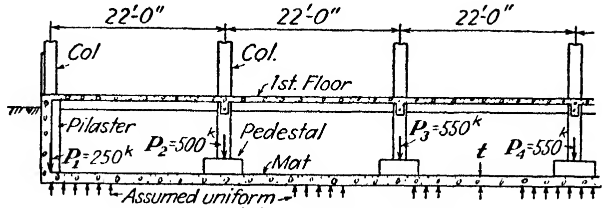
When the loads on adjacent columns (or in various areas of the basement) differ considerably, such a mat as this may be undesirable owing to the likelihood of local settlement because the mat cannot spread heavy loads very far over compressible soil. The bays in either direction should be reasonably equal in length, the larger spacing not exceeding 1.2 times the smaller one, and the columns should be arranged in reasonably straight rows. Furthermore, a spacing of columns exceeding 20 ft. may require an unduly thick mat.

One method of making a preliminary analysis of such a mat is on the basis of an assumed supporting system of column strips that constitutes a grid of beams along the column rows in each direction. If this grid is adequate, it can support the loads, assuming the portions of the slabs in the central areas to be supported by this grid.

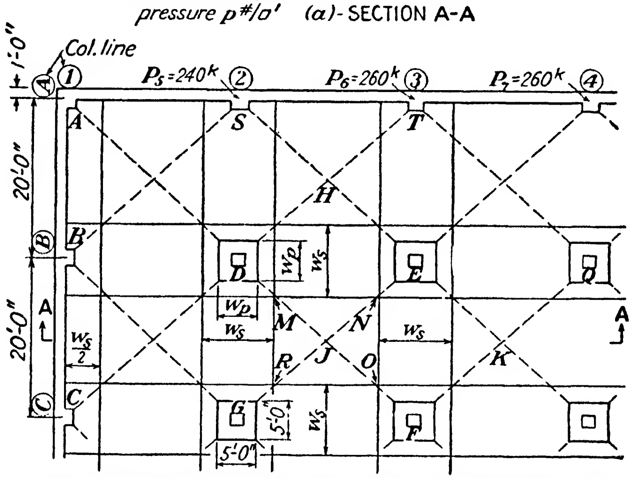
Referring to Fig. 7-1(*b*), assume that diagonal lines are drawn between the columns and that the pressure under area *DHEJ* is to be resisted by the portion of the mat between *D* and *E*. Assume also that the similar area *EKFJ* is to be supported by the mat between *E* and *F*. Next, assume that the reinforcement is so arranged as to form the chosen set of beams (column strips) extending each way along the column lines, as pictured by w_s in Fig. 7-1(*b*). The effective width w_s of these strips or shallow beams will have to be assumed. If the pedestals are thick enough to support the mat properly, w_s may be assumed to be wider than the pedestals, w_p. A minimum reasonable width may well be $w_p + 3d$ or $w_p + 4d$, where d is the effective depth of the mat. This gives an assumed strip a little wider than the width determined by 45° lines from the bottom of the pedestal to the lower reinforcement in the mat. Perhaps a wider strip might be assumed. At any rate, this requires an assumption of the thickness of the mat. If the effective depth d is assumed as equal to approximately 1 in. per ft. of clear span between pedestals for heavy loads (less for light loads), $d = 22 - 5 = 17$ in. Use this value and add 3 in. of cover. Then assume $w_s = 5 + 3 \times 1.42 = 9.3$ ft. Call it 10 ft. These column strips are to be designed strong enough by themselves to support the total loads.

Is this scheme sensible or wasteful? The answer depends upon the proportions of the pedestals, the spacing of columns, the loads, the concrete, and reinforcement needed to constitute the grid system. If the width of the pedestals is less than one-quarter of the column spacing, the column strips are probably a little too narrow.

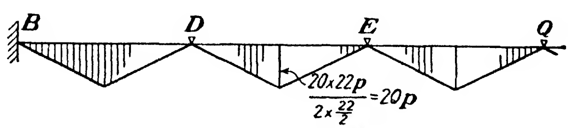
A central "suspended" panel like $MNOR$ of Fig. 7-1(b) may be designed as a two-way rectangular slab with fixed edges. When the sides MR and



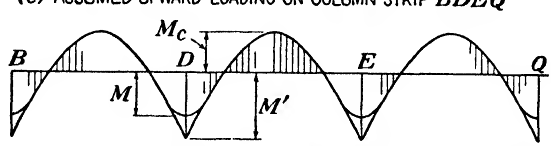
(a)-SECTION A-A



(b)-PLAN OF ASSUMED COLUMN STRIPS AND DISTRIBUTION OF LOADS



(c)-ASSUMED UPWARD LOADING ON COLUMN STRIP $BDEQ$



(d)-ASSUMED BENDING-MOMENT DIAGRAM FOR COLUMN STRIP $BDEQ$

FIG. 7-1. Assumptions for approximate analysis of a large mat.

MN are unequal, as in this case, the load will not be distributed equally in both directions, but it is a sufficient approximation to assume that it is so distributed when the dimensions are reasonably equal. Another assumption is that the load is divided between the two directions as shown by Fig. 2 in the Appendix. Assuming equal distribution, the bending moment per foot of width for the computation of the tensile reinforcement (at the bottom of the mat) crossing MN and RO may be assumed to be

$$M_{MN} = \frac{p}{2} \times \frac{1}{12} (MR)^2 = \frac{1.19}{2} \times \frac{10^2}{12} = 5.0 \text{ ft.-kips}$$

and

$$M_{MR} = \frac{p}{2} \times \frac{1}{12} (MN)^2 = \frac{1.19}{2} \times \frac{12^2}{12} = 7.2 \text{ ft.-kips}$$

The steel per foot of slab in the longer direction is then

$$A_s = \frac{7.2}{18 \times 0.9 \times 1.42} = 0.31 \text{ in.}^2$$

Use ⅝-in. rounds at 12 in. c.c. In the short direction these might be spaced at 15 in. All of this reinforcement should cross the full width of the column strip in order to get proper anchorage of the rods. At least 25 to 50 per cent of this bottom reinforcement should be extended across the central panels in order to have it wherever it may be needed. The remainder may be stopped, when theoretically not required, beyond the column strips.

Technically, the top reinforcement of a central panel may be less than that at the bottom. However, it may be advisable to reinforce both sides equally because any yielding of end restraint will cause the tension in the top of the mat near J to increase above the computed value. One-half of the top steel should extend to or somewhat beyond the edges of $MNOR$, then at least 25 per cent should extend across the column strips.

The purpose of the preceding requirements is to make sure that the mat is well tied together outside of and across the column strips which are indefinite in extent. Remember that assumptions *are* assumptions. The cost of the steel will be small; its service value will be great if need arises.

Assume that the column strip $BDEQ$ of Fig. 7-1(b) is to be analyzed. The loads acting upon it are assumed as the triangular loading diagrams shown in Sketch (c). The total load on each span is found from the applicable areas $DEFG$ and $DETS$ in Sketch (b), using the average for the two portions. Therefore,

$$p_{av} = \frac{1.19 + 1.11}{2} = 1.15 \text{ k.s.f.}$$

The total load on part DE is assumed to be the pressure acting upon area $DHEJ$. Therefore,

$$W = \frac{22 \times 20}{2} p = 220 \times 1.15 = 253 \text{ kips}$$

The mid-ordinate of the triangular pressure diagram for span DE is $253/11 = 23$ kips per lin. ft.

The column strip $BDEQ$ may be analyzed by moment distribution if the variations of loading or spans make this desirable, using the data in Fig. 7-2 for fixed-end moments. However, in this case, it is designed as a series of fixed-end beams. Member DE, for example, will have a triangular load of 253 kips on it. Then, from Fig. 7-2, the theoretical bending moment M' at one end, Fig. 7-1 (d), is

$$M' = 0.104 \times 253 \times 22 = 580 \text{ ft.-kips}$$

From Fig. 7-2, the central moment M_c is

$$M_c = 0.063 \times 253 \times 22 = 350 \text{ ft.-kips}$$

The bending-moment diagram is shown in Fig. 7-1(d).

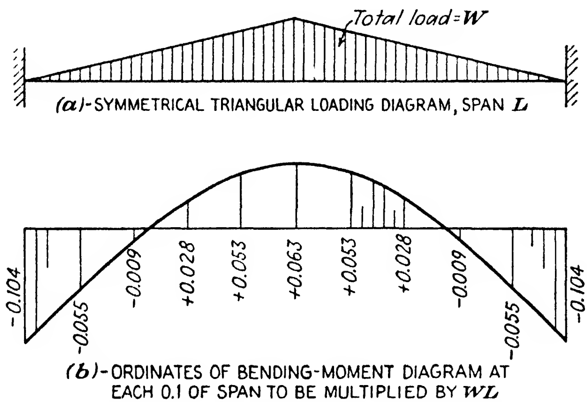


FIG. 7-2. Data for bending moments in rectangular beam with fixed ends. ***(Data from Hardy Cross and Portland Cement Association.)***

Should this bending moment at the supports be reduced because of the column and pedestal? The column will certainly spread its load over its area. If so, this will modify the bending-moment diagrams as indicated in Fig. 7-1(d). If the pedestals are deep enough, they will also spread the column loads, and they are assumed to be deep in this case. Therefore, the modified bending moment at point D, for example, is, approximately,

$$M = 580 - \frac{253}{2} \times 0.67 \times 2.5 = 370 \text{ ft.-kips}$$

where the downward force is applied at the center of gravity of one triangular quarter of the pedestal.

The reinforcement per foot of width in the bottom of the mat near the columns will be

$$A_s = \frac{1}{10}\left(\frac{370}{18 \times 0.9 \times 1.42}\right) = 1.6 \text{ in.}^2$$

Use 1-in. rounds at 6 in. c.c. At the top of the mat in the center of the bay, the reinforcement per foot will be practically the same since $M_c = 350$ ft.-kips. Therefore, the same reinforcement will be used.

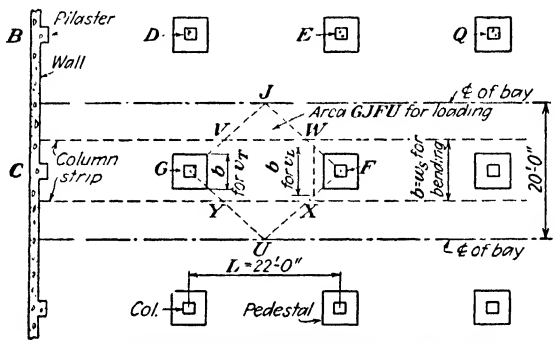
The punching shear at the face of the pedestal D is assumed to be

$$V_p = 1.15\left(\frac{5 + 20}{2}\right)(11 - 2.5) = 122 \text{ kips}$$

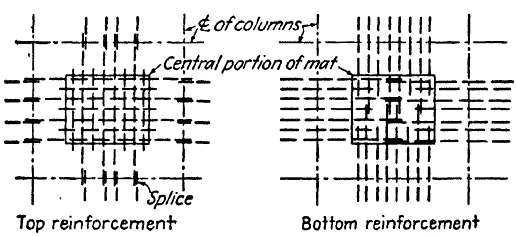
$$v_T = \frac{122{,}000}{60 \times 0.3 \times 17} = 400 \text{ p.s.i. (safe)}$$

As indicated in Fig. 7-3(a), the diagonal tension at a point 45° (1.42 ft.) beyond the bottom of the pedestal may be estimated as

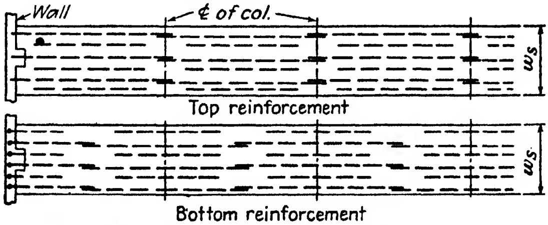
$$v_L = \frac{1{,}150\left(\frac{7.84 + 20}{2}\right)(11 - 3.92)}{(60 + 2 \times 17)0.9 \times 17} = 79 \text{ p.s.i. (safe)}$$



(a)-SUGGESTED ASSUMPTIONS FOR ANALYSIS OF COLUMN STRIP OF CONTINUOUS MAT



(b)-REINFORCEMENT IN CENTRAL PORTION OF MAT



(c)-REINFORCEMENT IN ONE DIRECTION IN COLUMN STRIP

FIG. 7-3. Some details of uniform, continuous mat.

Other spans, including the exterior bays, may be treated similarly. The general pattern of the reinforcement and the locations of splices are shown in Fig. 7-3.

Figure 7-5 shows the construction of a thick uniform mat that now supports a very heavy industrial building made of reinforced concrete. As indicated in the picture, the slab is embedded some 8 or 10 ft. so that the bottom story serves the purpose of a basement with conveyors and other equipment in it. The mat has a system of drains under and alongside it. The soil is a somewhat spongy volcanic material. There has been no harmful settlement of the structure.

It takes little imagination to realize that this mat will require a great quantity of concrete and that the large pedestals in the basement may be a hindrance to use of the space by the occupants of the structure. The first is inherent in the plan and perhaps unavoidable, but it should be judged in

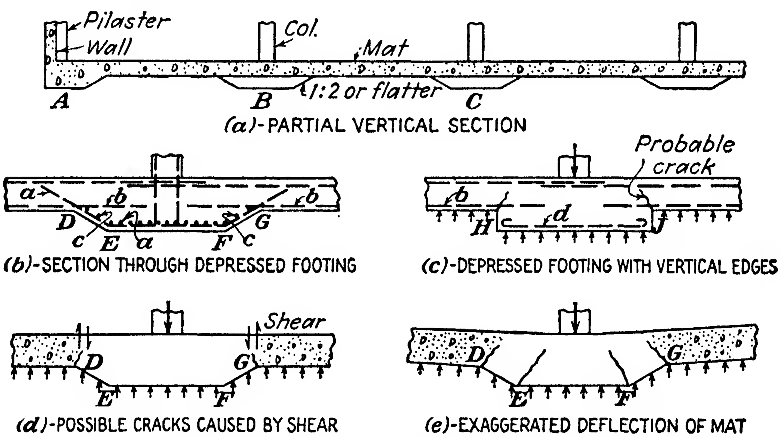


FIG. 7-4. Continuous mat with depressed footings instead of pedestals.

comparison to the cost of alternate types of foundation. The second might seem to be remedied by thickening the slab locally at the bottom, as pictured in Fig. 7-4(a). If the thickness under the column is to be great enough for punching shear, if the compressive stresses in the footing or mat around the column are to be reasonable, and if the tensile reinforcement under the column is to be moderate, the thickening under the column must be very considerable. This requires the careful excavation of a pocket at each column as indicated by $DEFG$ of Sketch (b).

It might seem that the reinforcement of this mat could be arranged as indicated in Fig. 7-4(b), having rods a and b lapped as shown. This might be done for the rods in the flat portion, as shown by EF for the rods at 90° to the cut section. However, the rods c in the sloped portions such as DE and FG will have varying bends, and all of them make a complicated situation where a and c in both directions intersect rods b which are also in both directions. When the mat beyond D and G is rather thin, rods a and c may have to be hooked or bent in order to develop bond properly. This complicates the reinforcement still more. If rods a and c are bent down parallel to and alongside b, their tendency to straighten under tension may spall the concrete and cause failure of the mat.

The sloped sides DE and FG of Fig. 7-4(b) are made this way in order to ensure good bearing on the soil. Assume that the footing is made as in (c) with a steep offset. The soil near H and J is likely to be weakened. Furthermore, the upward shearing forces at H and J caused by the mat are

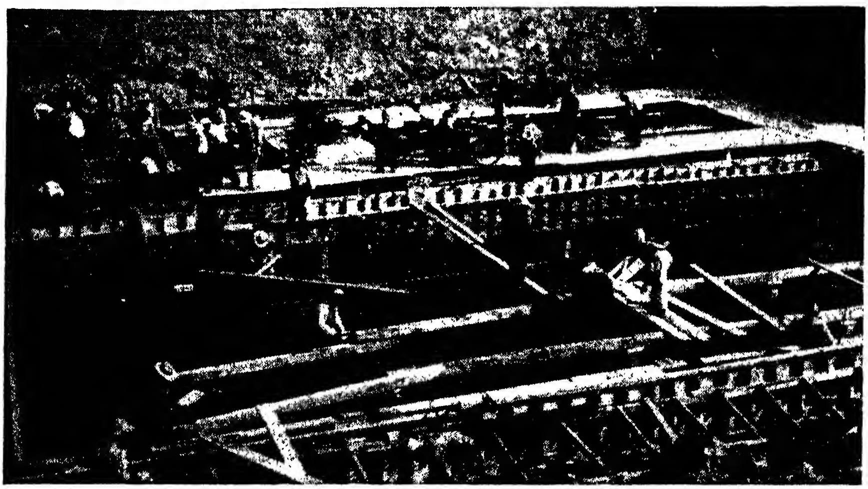


FIG. 7-5. Construction of a heavy mat under a secondary crushing plant. The soil is of volcanic origin and is somewhat spongy. *(Courtesy of the Cananea Consolidated Copper Co., S.A.)*

likely to crack the structure as shown. Even though rods d are bent vertically upward and hooked into the top of the mat, their effectiveness is questionable. Concrete construction can stand pressures easily; when it must be stitched together with reinforcement to prevent being pulled to pieces, the advisability of the basic plan is questionable. Even with the sloped bottom, there is still the tendency to load the edges of the thickened section as illustrated in Sketch (d). An exaggerated picture of possible failure is shown in (e).

Another variation of the construction is the use of enlarged strips shaped like the section shown in Fig. 7-4(b). These are to extend continuously in both directions under the columns. Thus they form a grid of column strips or beams that will act even more closely in accordance with the assumption that the column strips constitute a system of beams that supports the entire pressure under the mat. In effect, it is a checkered pattern of shallow wide beams with thin central panels attached to them. The excavation is somewhat more troublesome than is that for a uniform mat but this is usually of little consequence. This system may be very useful.

In Fig. 7-6 is shown another arrangement for a continuous mat.[1] This general scheme was used, for example, in the Kansas City, Kan., plant of Sunshine Biscuits, Inc. Beneath the structure is about 12 ft. of gumbo, an alluvial deposit of clay, silt, and fine sand. The grillages and welded column stubs shown in Sketch (*a*) serve as a sort of footing to spread each column load over a sufficiently large area of the mat to avoid excessive bending, shear, and diagonal tension, with the help of the U-stirrups when necessary. In this way, heavy billets under the steel columns are avoided,

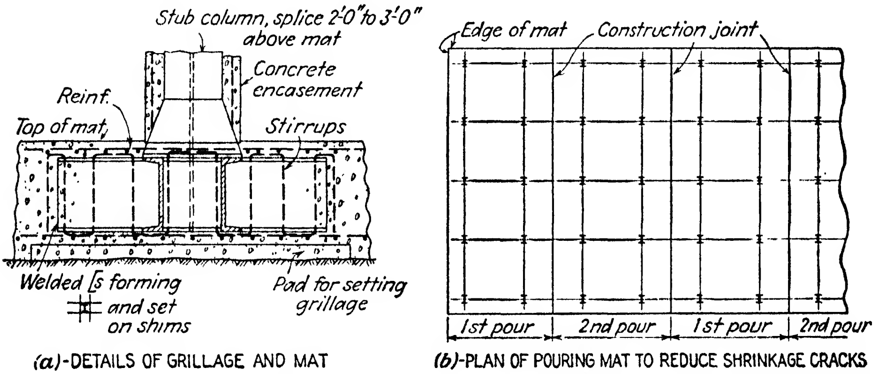


FIG. 7-6. Patented system using steel grillages to spread load into uniform mat. (Courtesy of Walter H. Wheeler, Minneapolis, Minn.)

projections above the floor are prevented without increasing the thickness of the mat, and a slab of uniform and moderate depth can be used, thus facilitating preparation of the subgrade.

Many other details are shown in Fig. 7-6. Special attention should be given to the following features:

1. The assembly of the grillage on seat angles and a concrete pad, with space for the reinforcement to be inserted.

2. Most of the load from the column reaches the concrete through the bearing of the top flanges of the channels, although bond may help considerably also. Notice that downward pressure on the bottom flanges of the channels will tend to spall off the concrete whereas the stirrups tie back the bottom to the main body. Proper design of these grillages is important.

3. The general arrangement of the reinforcement. The principles of design are similar to those used for flat-slab floors.

4. The construction joints, shown in Sketch (*b*). To minimize shrinkage, the slab was first poured as bands in alternate strips across the width of the

[1] Patented by Walter H. Wheeler, Minneapolis, Minn. See Multistory Factory Floats on Gumbo, *Engineering News-Record*, Mar. 3, 1949.

building, then the remainder was filled in after initial shrinkage occurred. The concrete was made with low-alkali content, type II cement to reduce shrinkage in general.

A system of underdrainage was installed to remove ground water to sumps and automatic pumps. This system was to prevent hydrostatic pressure under the slab. It works well in impermeable soils, but porous materials tapping large sources of ground water might require too much pumping.

A mat is a required structural member used to spread loads. There are cases where the construction appears to be a mat but is not one. For example, the bottom of a concrete tank placed directly on the ground may have a continuous concrete floor. This, however, is not a structural mat to distribute concentrated loads; it is a pavement to prevent leakage. Such a slab may well be only 4 to 8 in. of concrete or gunite, but it should be strongly reinforced, perhaps with a steel area of at least 0.5 to 1 per cent of the cross-sectional area of concrete, and with the rods extending in both directions. In such a case, the slab is purposely made thin and flexible so that it can accommodate itself to unequal settlements of the ground without too harmful cracking. It has no influence upon the pressure under the tank or the settlement of the soils below it.

7-3. Ribbed mat. The preceding article shows, when one examines the bending moments in the mat, that it may be desirable to strengthen the grid of assumed supporting slabs by using a system of heavy beams or walls along each column row and in both directions, then to design the mat as a system of two-way slabs supported by these beams. Of course, these beams will generally have to be very heavily reinforced for bending and shear. This arrangement has many obvious difficulties. If the beams are deep ribs placed below the basement floor or mat, as in Fig. 7-7(*a*), the bottom of the excavation becomes badly cut up with trenches, the work will be costly, and the bearing value of the soil may be impaired because of disturbance of the ground next to the ribs. If the construction is inverted so that the mat is at the bottom, the ribs will destroy the usefulness of the basement unless the main foundation is depressed and a basement floor is placed over it, somewhat as shown in Sketch (*b*). On the other hand, walls with doorways through them might be used as indicated in (*c*) if a series of small rooms is acceptable--which is seldom the case. Any of these grid systems will generally distribute a moderate variation in column loads satisfactorily if the detailed design of the ribs provides for this. Any such construction should be strong and stiff. It is a poor place to skimp in the use of steel and concrete. Furthermore, the owner seldom consults the designer when he intends to increase the loads applied to the structure at some future time.

An alternate method that may be used sometimes with a continuous mat under a basement is a system having continuous walls (with occasional doorways) extending along the column lines in one direction only. A possible arrangement for this construction is shown in Fig. 7-7(d). Since the mat is primarily a heavy one-way slab, it may be desirable to thicken it as indicated in the sketch in order to have greater resistance to bending and shear at the walls and to economize on the use of concrete in the central region of the spans. The walls are usually so deep and strong that, if the

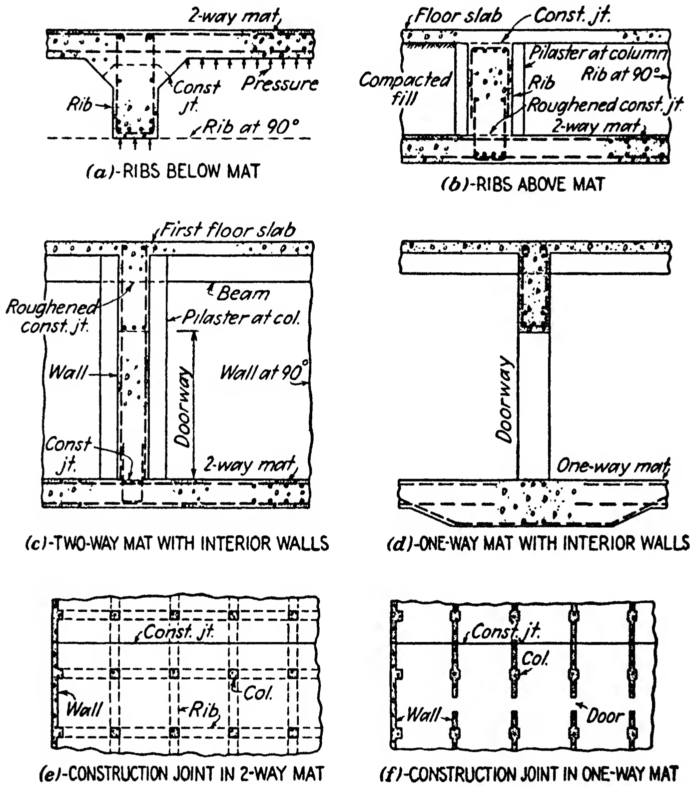


FIG. 7-7. Some construction for ribbed mats.

doorways are at the centers of the bays and if they are not more than 4 or 5 ft. wide, one may assume that the column loads are distributed uniformly along the walls, including the space below a doorway. As this system does not have great strength to distribute loads in a direction perpendicular to the walls, the column rows should be loaded reasonably equally.

In Fig. 7-7, the general arrangement of the principal reinforcement only is shown. The slabs and ribs or walls should be designed as continuous members. Of course, construction joints are necessary. Suggested locations for these between ribs and mats are pictured in (a) and (b). If vertical construction joints are to be used also, it is generally desirable to locate them at points of small shearing forces, as indicated in Sketch (*e*). When vertical construction joints are to be in a one-way mat construction, it is preferable to locate them at the centers of bays in which there are no doorways in the walls. Horizontal keys in the mat and intermittent keys in the ribs and walls are desirable at such joints.

The ribs and walls of a rectangular grid system such as that of Fig. 7-7(*e*) may be analyzed as fixed-end or continuous beams with triangular loading based upon the principles of distribution shown in Fig. 7-1(*b*) for column

FIG. 7-8A. Ribbed-mat substructures of the Central Mill Boiler House, Anaconda Wire and Cable Company, Hastings-on-Hudson, N. Y. The interior columns will rest upon the ribs at their intersections. The cantilevered portion of the mat will be covered with earth.

strips. The reaction points are at the columns and outside walls. The slabs may be proportioned to resist uniformly distributed loads as two-way slabs with fixed or continuous edges.

The walls of a one-way mat system like that of Fig. 7-7(*f*) may be analyzed as continuous or fixed-end beams loaded uniformly along their lengths, with reaction points at the columns. If the walls are cut by relatively small shallow doorways at the centers of the bays, these openings may be neglected; if the doorways are large or high, it is best to consider the wall as hinged at these points as far as bending is concerned but to reinforce the top and bottom of each opening enough to prevent cracking. The mat, of course, is to be designed as a one-way, fixed-end, or continuous slab uniformly loaded by the net upward pressure.

A two-way mat with ribs for a small boiler house is shown in Figs. 7-8 and 7-9. The general dimensions of some of the construction are given in Fig. 7-9 in order to show some scale regarding expected sizes for net pressures of approximately 1,000 p.s.f. with considerable spreading of the heavy loads required from the central boiler columns toward the outside.

A brief outline of the evolution of this design may be instructive:

1. At the first site selected near the river, the borings showed approximately 20 ft. of fill, a tapered stratum of some 40 ft. of soft clay, a few feet of silt, some fine sand, and then coarse sand. It was decided that 90-ft. concrete-filled pipe piles should be used to support the structure.

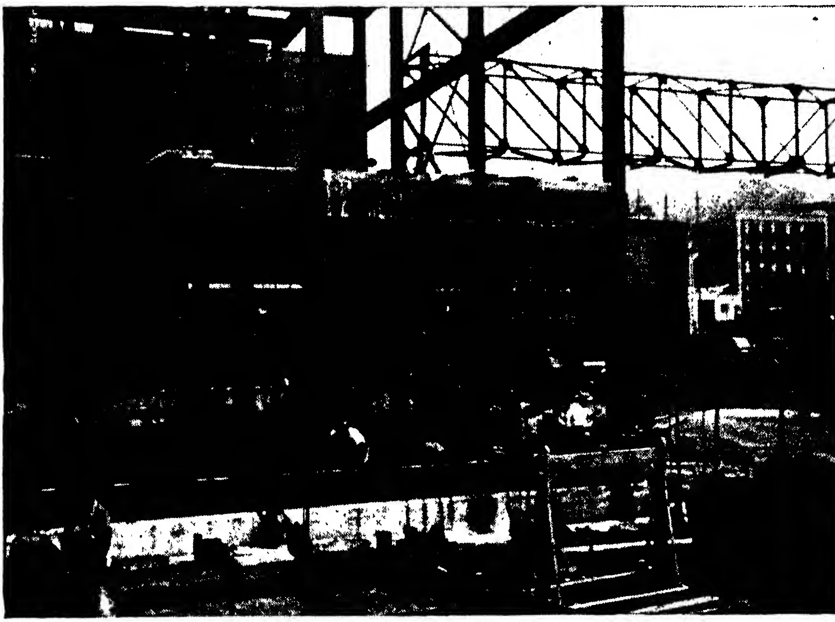
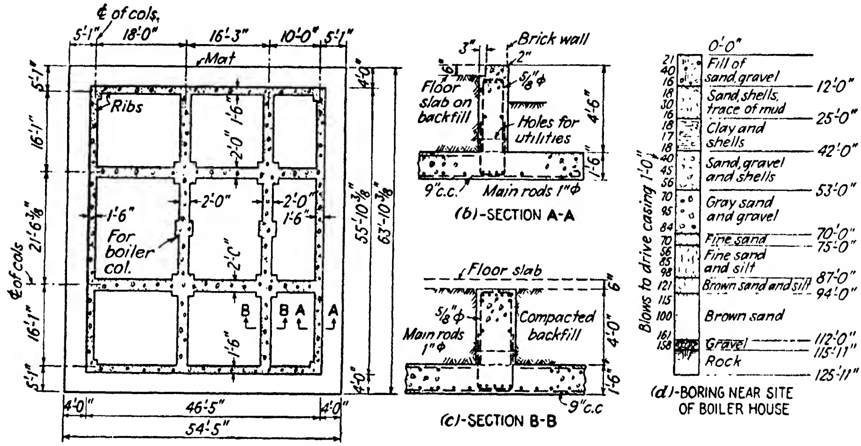


FIG. 7-8B. The structural framework of the Central Mill Boiler House, and the construction of the brick walls. The floor slab was not poured until after the installation of equipment and piping. *(Courtesy of Anaconda Wire and Cable Company.)*

2. An alternate shoreward location was investigated. The old fill here was only about 12 ft. deep, the clay stratum was firmer and only about 15 ft. thick, fine sand underlay it, and then there was coarse sand at a depth of 40 ft. It seemed to be safe to use a mat foundation here and thus avoid the expense of using piles. The ribbed type was adopted in order to spread the interior column loads.

3. The design for the superstructure is shown in general in Fig. 7-10. The two small wings at the right and left illustrate a principle to be borne in mind when planning mat foundations: the desirability of fairly uniform loading or of great stiffness of the structure in order to spread the loads. The central loading here is heavy, whereas the weight of the wings is light. As the structure settles—and some compaction of these soils is inevitable—

there would be a tendency for the center to settle more and to shift loads to the wings, probably breaking the substructure near the outer walls of the main building because of the upward bending moments. The entire mat will try to act as a unit. One should not assume that the ground under one part can be loaded heavily whereas the remainder is loaded lightly unless



(a)-PLAN AND HORIZONTAL SECTION THROUGH RIBS

FIG. 7-9. Ribbed mat for Central Mill Boiler House, Anaconda Wire and Cable Company, Hastings-on-Hudson, N. Y.

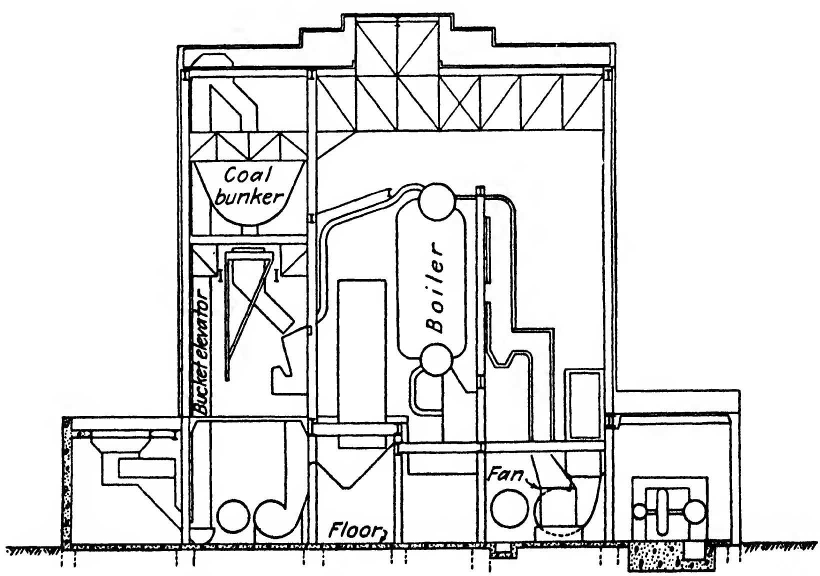


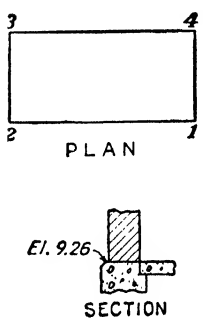
FIG. 7-10. Preliminary study of small boiler house.

unequal settlements occur. Therefore, the layout was revised to eliminate the extensions and place the pumps and compressors inside the main walls.

Level readings to show the settlement of this structure were taken once a month for over 2 years. The results are given in Table 7-1. As expected, settlements occurred, but they seem to have been reasonable and to have been substantially completed in 2 years.

When deciding upon such important matters as the use of a mat or of a deep more costly substructure, one should not forget that there is little to be done if the future proves that this decision was unsound.

TABLE 7-1. Settlement Record of a Small Boiler House Supported by a Ribbed Mat on Clay and Sandy Strata

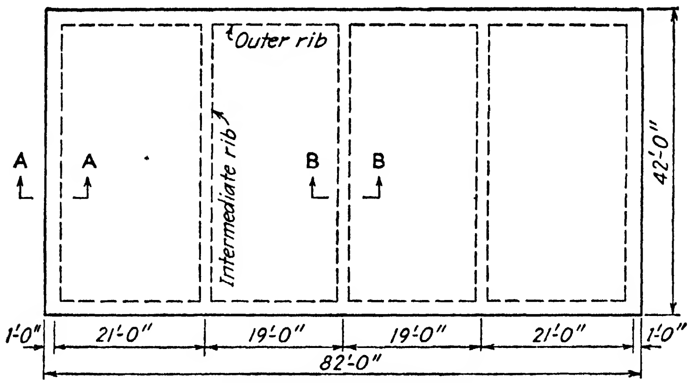


Date	Sta.	Δ	Date	Sta.	Δ
Jan. 14, 1947	1	3/32	Aug. 21, 1947	1	9/16
	2	5/8		2	1 5/16
	3	23/32		3	1 11/16
	4	13/32		4	27/32
Feb. 12, 1947	1	1/8	Oct. 17, 1947	1	17/32
	2	25/32		2	1 11/32
	3	31/32		3	1 11/32
	4	17/32		4	27/32
Apr. 17, 1947	1	3/8	June 1, 1948	1	19/32
	2	1 1/32		2	1 9/16
	3	1 1/32		3	1 11/32
	4	23/32		4	27/32
June 26, 1947	1	1/2	Mar. 24, 1949	1	5/8
	2	1 5/32		2	1 3/8
	3	1 7/32		3	1 11/32
	4	27/32		4	1 3/16

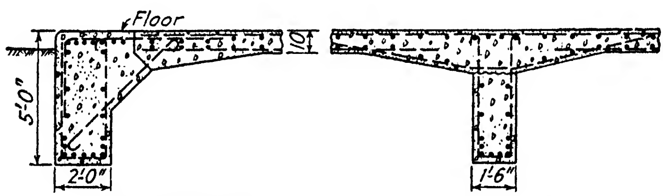
Δ = settlement in inches below El. 9.26

Another case is illustrated by a small laboratory building that was to have brick walls and a steel trussed roof. It was to be founded upon a 20-ft. blanket of new fill placed over soil of questionable bearing value. It did not seem to be worth while to drive long piles to hold up the structure. Therefore, the design shown in Fig. 7-11 was developed. In principle, it is somewhat like an inverted concrete barge with a continuous mat as a floor and with heavy ribs around the edges and across the width. The substructure is planned so as to have sufficient rigidity to prevent local sagging. It was expected that the entire structure would settle a little. This, however, has not been noticeable, and no objectionable cracks have appeared in the concrete or the brickwork.

This construction in Fig. 7-11 proved to be so successful that the designers used the same scheme for a 60 by 240-ft. clear-span structure that was to be supported upon new but good fill that varied from 9 to 14 ft. in depth. The intermediate cross ribs were omitted. Columns were along the outside only. The men in the field proposed that concrete piers with footings be used at 15 or 20 ft. c.c. under the peripheral ribs. These would have been



(a)-PLAN OF SUBSTRUCTURE

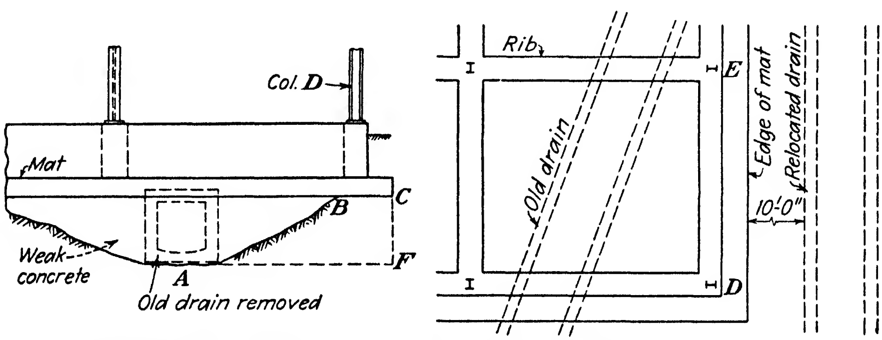


(b)-SECTION A-A (c)-SECTION B-B

FIG. 7-11. Construction of ribbed mat for small industrial building.

harmful for the structure because the edges could not settle with the fill, whereas the central part of the floor would have to do so because the slab could not span a distance of 60 ft. safely. The basic idea of the design was that it should accompany the settlement of the fill without injury. If piers were used around the edges, it would be better to cut the floor slab loose from the outer wall so that it could settle with the fill. This, however, might cause trouble at doorways and partitions.

When planning any mat foundation, the engineer should explore the site thoroughly to make sure that conditions are suitable. It is very important to avoid relatively hard spots and soft spots under the mat. For example, plans were made for a mat foundation under an industrial building that was to be placed close to an existing structure. The main foundation consisted of an 18-in. slab with a heavy grid system of rib walls around the outside of and across the mat along the column lines. When the excavation was made, a forgotten old box drain was encountered, somewhat as shown in Fig. 7-12(a). The drain cut across one corner of the area for the mat, as pictured in Sketch (b). The contract had been let, and it was too late to make extensive changes in the plan. The old drain was therefore removed, and a by-pass was built as indicated, then the trench was cleaned out, and the sides were sloped to avoid leaving weakened material alongside. If the space thus made were filled with weak concrete, it seemed that there might be a wedging action along the sloped surface AB, causing the soil to move from under AB and BC. If so, the column loads at D and E



(a)-PARTIAL VERTICAL SECTION (b)-PLAN OF CORNER OF SUBSTRUCTURE

FIG. 7-12. Unexpected difficulties caused by inadequate preliminary exploration.

might cause the structure to "break its back" over the ridge A. It was therefore decided that the earth outside of AB should be excavated down to the level shown by the dotted line AF, and that the entire space below the mat should be filled with weak concrete. Even then, the situation is one to worry an engineer. Were the extra excavation and concrete in the space $FABC$ necessary? Remember it is not wise to wait and see what happens before applying a remedy. If the drain had been discovered before the design was made, the mat should have been lowered to avoid this bulge, or it might have been sloped gradually upward away from F and A.

A surprise developed in the use of a mat with ribs like those of Fig. 7-7 for a boiler house. This may well serve as a warning. One of the rectangular spaces between the upstanding ribs or walls near the center of the structure was used as a sump. The engineers started testing the boilers one winter day when the temperature outside was below freezing. After the test, they emptied boiling water into the sump. A short time later, the cold exterior walls opposite the sump were found to be cracked noticeably. Apparently the expansion of the central portion of the concrete forced the cold outer concrete and the steel of the narrow side to crack because the former acted like a tremendous jack.

When working on the foundation of a power plant or any structure that has equipment for generating electricity, one should consider the possible action of the reinforcement in conducting stray currents of electricity. Not only may this cause loss of power, but the electrolysis may damage the steel. Separation of various layers of the mat reinforcement and the splices of bars by an intervening 2 or 3 in. of concrete may be helpful.

7-4. Mats to resist hydrostatic pressure. In the planning of basements, pits, depressed tanks that may be emptied, and other structures that are to be watertight, one frequently encounters the problem of resisting hydrostatic uplift. The soil may be satisfactory sand or gravel so that the downward loads on columns and walls may be supported by footings; nevertheless, the water pressure causes upward forces that affect the floor somewhat as do the soil pressures discussed previously. The case may be complicated, however, by buoyancy.

Example. To illustrate some of the problems involved in the planning and detailed designing of such foundations, assume the simplified case shown in Fig. 7-13. A manufacturer is to build a new plant near tidewater. Under one of the crane aisles he wishes to have a deep pit with certain machinery in it. The soil, the column loads, the weights of the machines and their pedestals, and the probable maximum elevation of the watertable are shown in Sketch (*b*). How may the pit and the adjacent column foundations be designed? The safe bearing value of the soil is 3 tons per ft.^{2} Assume that pumps and wellpoints are to be available to lower the watertable temporarily so that construction work may be carried on "in the dry."

In attacking this problem, the following assumptions may be made, basing them upon the situation shown in Fig. 7-13:

1. The bottom and sides of the pit are to be covered with three-ply membrane waterproofing.

2. Columns *G* and *H* will be so close to the pit that their foundations may be incorporated in the construction of wall *CF*, or they may be immediately outside of this wall, whichever seems advisable.

3. When placed below the stratum of silty clay, the footings for columns *J* and *K* will be too low to cause harmful effects on the pit construction.

4. The floor slabs surrounding *CDEF* will be seated upon and tied to the pit walls in order to act as horizontal beams that will support the tops of the pit walls against the horizontal pressures produced by the surrounding soil and water.

5. Since the elevation of the watertable is not a completely known and trustworthy figure, the pit walls and floor will be designed to resist lateral and upward pressures based upon the tentatively determined maximum height shown, letting any excess caused by a serious flood be taken care of by the safety factor provided in the customarily used unit stresses in the concrete and steel.

6. For resistance to buoyancy, the pit will be designed to resist hydrostatic pressures caused by water up to the floor line, the point of flooding. The downward weights to be assumed for equipment A and B will be the weights without the machines themselves. Similarly, the minimum dead-load reactions only of columns G and H will be relied upon, if they are supported upon the pit construction. The safety factor to be attained under this condition need be only slightly greater than unity. Since the access stairway is to be bolted steelwork, to permit future rearrangement, its weight is small and will be neglected.

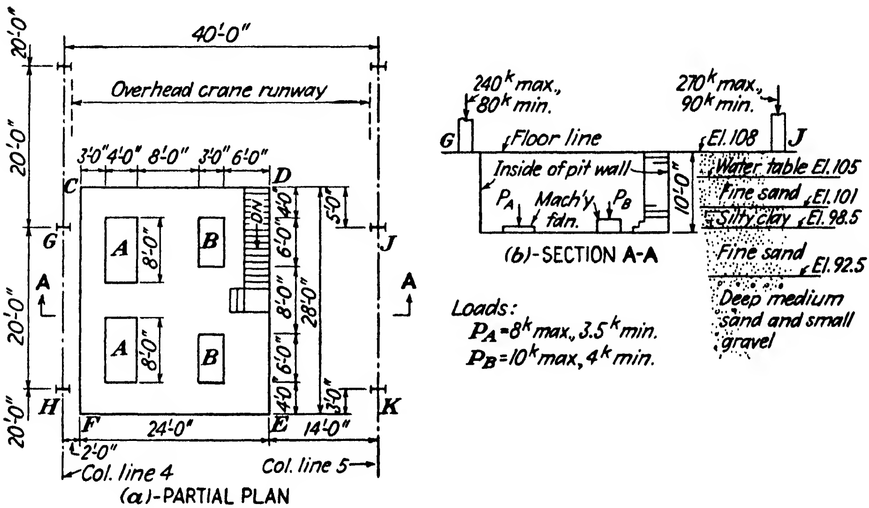


FIG. 7-13. Layout showing large pit to be built as part of an industrial plant.

7. The combined horizontal unit pressure caused by the soil alone will be assumed equal to 30 p.s.f. per ft. of depth; that of saturated soil, 80 p.s.f. per ft. of depth below the watertable.

8. The upward unit hydrostatic pressure will be assumed equal to 63 p.s.f. per ft. of depth below the surface of the water.

To start the solution of this problem, first obtain a general idea of the seriousness of the buoyancy problem. Assume the side walls to be 12 in. thick; the bottom, 12 in.; the upper floor slab for a 10-ft. width around the pit walls, 8 in. Columns G and H do not rest upon the pit.

The maximum weight of displaced water when flush with the floor is found as follows:

Pit = $26 \times 30 \times 11 \times 63$	540,000
Upper floor = $(46 \times 50 - 26 \times 30)0.67 \times 63$	64,000
	604,000 lb.

The minimum downward force exclusive of columns G and H is

Bottom $= 26 \times 30 \times 150$	117,000
Walls $= (48 + 60)10 \times 150$	162,000
Upper floor $= (46 \times 50 - 26 \times 30)\dfrac{100}{2}$	76,000
Machine bases	15,000
	370,000 lb.

It would seem that the dead loads of columns G and H may be used to help counteract the excess of upward over downward forces. However, if they and the essential masonry under them are included, the center of gravity of the downward forces will be nearer wall CF than will that of the upward forces, thus causing a rotation about that edge and a tendency to lift wall DE. Furthermore, it would be wise to make sure that, after the pit is built and the pumps removed, a flood cannot cause trouble before the superstructure is completed.

How can the deficiency of weight be overcome, and what remedy will be both reliable and economical? Here are a few possibilities:

1. Might it be necessary to increase the thickness of the pit floor to withstand the bending moments, thus securing more weight automatically? The net upward pressure under the floor for design is

$$p = (11 - 3) \times 63 - 150 = 350 \text{ p.s.f. (approx)}$$

Using Fig. 2 in the Appendix, p' for the 24-ft. direction $=$ 200 p.s.f.; p'' for the 28-ft. direction $=$ 150 p.s.f. (approx)

$$M_{24} = \frac{200 \times 24^2}{8} = 14{,}400 \text{ ft.-lb. (counting no end restraint) for a 12-in.}$$

strip.

$$M_{28} = \frac{150 \times 28^2}{8} = 14{,}700 \text{ ft.-lb. (approx)}$$

$$f_c = \frac{14{,}700 \times 12}{0.3 \times 0.9 \times 12 \times 9^2} = 670 \text{ p.s.i.}$$

$$A_s = \frac{14{,}700}{18{,}000 \times 0.9 \times 0.75} = 1.2 \text{ in.}^2$$

It seems that, from the standpoint of strength, a 12-in. slab is sufficient. However, it might be made thicker to obtain more weight, but this increases the excavation and the pressure on the invert, as shown in Fig. 7-14(a).

2. The side walls might be thickened as in (b) but this, too, gains only about 90 p.s.f. for each cubic foot of concrete used.

3. The pit floor might be cantilevered outward like a footing, as in (c), so that the weight of earth on top of it may be included. If the soil weighs 100 p.c.f. and solid stone weighs 165 p.c.f., the volume of solids in the soil

per cubic foot is $100/165 = 0.61$ ft.[3] Therefore, the unit weight considering buoyancy is $100 - 0.61 \times 63 = 62$ p.c.f. Such a projection will hit the foundations of columns G and H. The presence of the membrane waterproofing, however, makes it difficult to transmit the net weight of the earth to the pit construction unless the membrane encloses the projections. The water must be assumed to penetrate to the outside of the membrane itself, and keys, dowels, shelves, and such shear locks or ties may cause leaks in the membrane.

4. The main floor around the pit might be thickened, as in (*d*), but buoyancy and the fact that at least half the weight of this floor will rest upon the surrounding soil render this method less economical than some of the others.

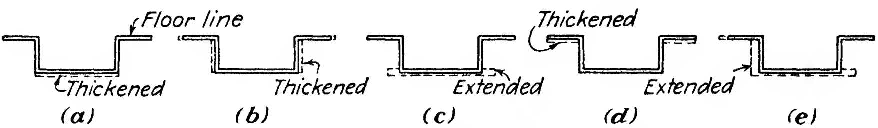


FIG. 7-14. Methods of increasing weight of pit construction.

5. Ribs or walls extended from DE to the foundations of columns J and K in order to utilize their weight would have to be very strong and would be outside of the waterproofing. This might be a good way to obtain extra weight if the columns were in line with walls CD and EF. However, for the situation as it is, this remedy may not be advisable.

6. It is probable that frictional forces and a downward component caused by the lateral pressures exerted by the soil will offer resistance to upward displacement of the pit. These, however, are not absolutely trustworthy in their action, especially because they are applied outside of the rather frictionless surface of the membrane waterproofing.

The most trustworthy and practicable solution seems to be the following, which is shown in Figs. 7-14(*e*) and 7-15:

1. Increase the pit floor to a thickness of 15 in.
2. Increase wall CF to 3 ft. 6 in., and use it as a foundation wall for columns G and H.
3. Cantilever the pit floor as a footing beyond sides CD, DE, and EF.
4. Arrange the details of the footing and wall construction as shown in (*c*). Although the excavation is increased somewhat and the waterproofing of the projecting footings is more troublesome, these matters are not too serious.

An approximate computation of the *added* weight to resist uplift is the following, assuming 87 and 62 p.c.f. as the net unit weights of concrete and earth, respectively:

Pit floor and footings:

$$(31 \times 36 \times 1.25 - 26 \times 30 \times 1)\ 87 = 55{,}000 \text{ lb.}$$

Wall $CFLM$:

$$(3.5 - 1)30 \times 10 \times 87 = 65{,}000 \text{ lb.}$$

Earth on footing (minimum):

$$(2 \times 31.5 + 30)3 \times 9.33 \times 62 = 161{,}000 \text{ lb.}$$

Total added weight = 281,000 lb., which exceeds 604,000 − 370,000 = 234,000 lb., the former deficiency. This is safe without relying upon the

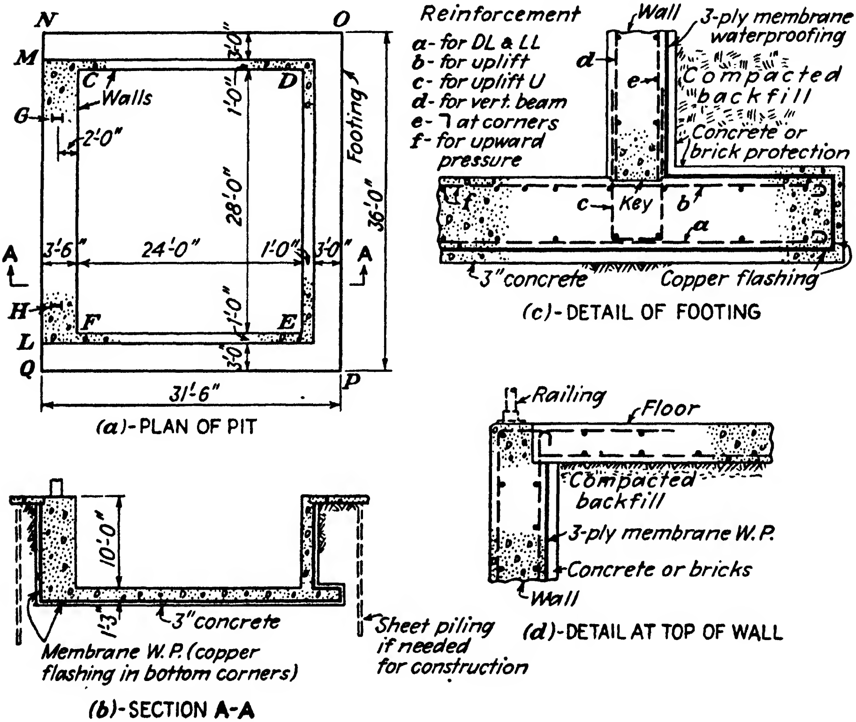


FIG. 7-15. Proposed construction for large pit.

dead load of columns G and H. If the weight on these columns is utilized, the cantilevering of the footing might be decreased, but the resultant economy will not be large because one must be careful that the side DE cannot lift up.

After this general design has been approved, the analysis of it may be completed by the reader.

7-5. Floating foundations. There are occasionally situations in which it is inadvisable to apply any major increase of pressure to the soil at a specific site. One principle that may be utilized, however, in such cases is that of the "floating" foundation. This means that the estimated weight of soil, removed permanently in order to build the substructure, should equal the load to be applied later by the entire structure. It is reasonable to expect that serious settlement will not occur unless additional pressure is applied to the soil, basing this belief upon the supposition that the soil has been subjected to the present pressures for so many years that, for practical purposes, equilibrium has been reached and further consolidation would be too slow to be important.

For example, assume that the total dead load of a building equals 1,500 p.s.f. over the horizontal projection of the structure. If a basement with some type of mat construction is then built under the entire structure and if the bottom is set approximately 15 ft. below the surface, the lower strata should be unaffected. As the excavation is made, there will be some elastic springback of plastic soils, but this is not likely to result in serious recompression. This procedure naturally implies that the center of gravity of the applied loads should coincide closely with that of the mass of earth excavated. In computing the weight to be supported, the live load may usually be neglected, except possibly for the inclusion of 25 or 50 per cent of that to be expected in large warehouses.

The planning and detailed designing of the substructure will be similar to those of other basements supported upon a mat. Waterproofness will generally be essential. Without this quality, seepage and staining will probably cause the owner to be dissatisfied with the results even though the building is structurally safe. The effect of the uplift caused by a varying elevation of the watertable will seldom be harmful if the depth of the excavation is determined upon the assumption of a low water level whereas the detailed design of the structural side walls is based upon the pressures caused by flood conditions. Since the dead load of the building is constant, it makes no difference to the mat whether the upward pressure is that of the soil alone or of soil and water acting in unison.

7-6. Examples of planning. The reader has been and will be cautioned repeatedly to look upon a structure as a whole when he plans the foundation for it. A minor case that illustrates the psychological side of such matters is shown in Fig. 7-16. Sketches (*a*) and (*b*) show the original plan for a compartmented concrete bin for the storage of steel and cast-iron balls for use in the ball mill (grinding equipment) at a metallurgical plant. It was located where three very heavy building columns came along one wall and some light columns for an elevated platform rested on the opposite wall. All six columns were planned to have concrete piers that extended from the top of the bin to footings under its bottom. These piers had keyways to tie them to the bin walls laterally, but they were to be separated from them by joints as shown so that the walls would not participate in resisting the column loads, and vice versa.

Referring to Fig. 7-16, notice the following points:

1. Sketch (*b*) shows the footings for the *A*-line columns to be under a part of the bin floor so that the same area of ground is forced to resist both column and heavy bin loads. The excavation for the footing will almost inevitably weaken the bearing value of the ground under the bin just beyond the edges of the footing.

2. The joints between the piers and the walls are so deep that freedom of action is almost impossible to attain without the use of some proper type of joint filler.

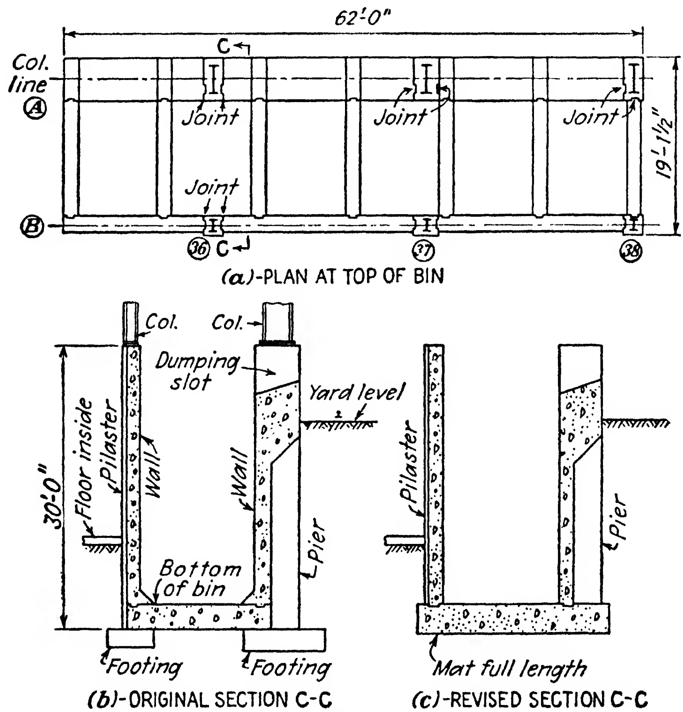


FIG. 7-16. Construction of a concrete bin and column foundations.

3. The end wall of the bin at column line 38 is not tied to the side walls to keep them from spreading.

4. The bin will be heavily loaded at some times but empty at others, and any individual compartment may be filled. Because of the heavy live loads, the reactions on the building columns will also vary greatly and rapidly. There will be no coordination between these loadings. This will tend to injure the structure.

5. The long sides of the bin are cut into isolated sections. At column lines 36 and 37, the walls act as cantilevers to resist bin loads, and the walls must steady the piers under the columns.

6. There are very heavy transverse shears on the columns in line A. That is why they are made so deep. These shears must be resisted by pieces of the bin walls, an action which is undesirable.

7. The construction work itself will be unnecessarily complicated.

Figure 7-16(c) shows the changes that were made to remedy these troubles. The bin was designed as a boatlike structure with a heavy mat

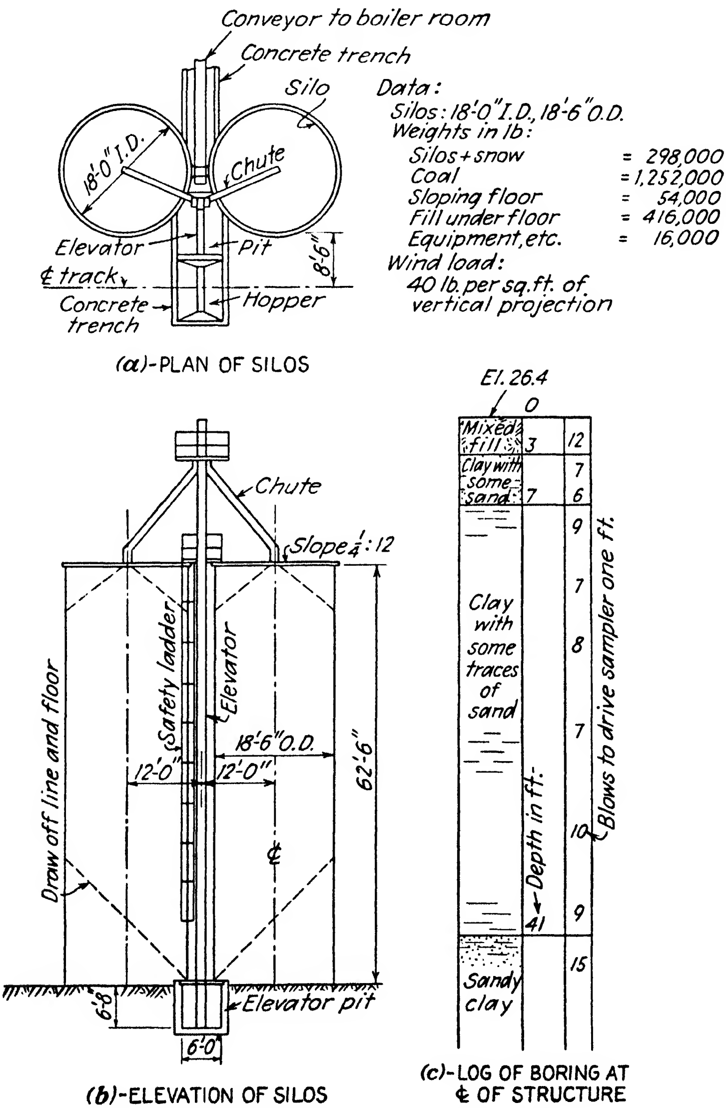


FIG. 7-17. General design for two coal silos.

under it and cantilevered outside of it enough to enable it to spread the loads safely to the soil no matter what the combination of live loads might be. The wall along column line A is in effect an elongated foundation for the heavy columns. The bin is now tied together thoroughly, and construction joints are omitted except for the horizontal ones between the monolithic pours. The mat was protected by a sand cushion above it. Nevertheless, it was made excessively thick to prevent serious consequences from wear. The structure now acted as a unit, as it would try to do anyway.

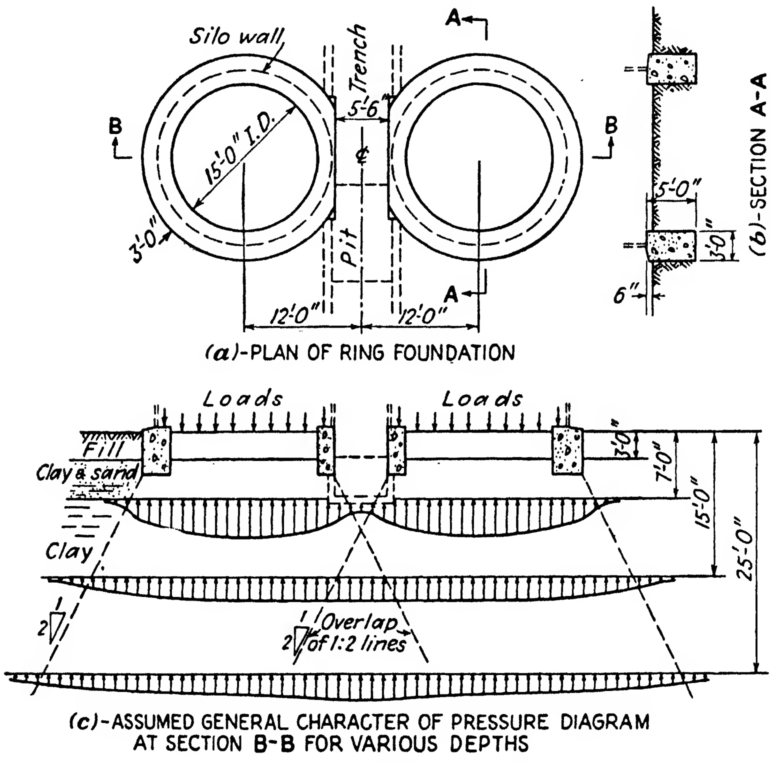


FIG. 7-18. One study of foundations for two coal silos.

Figure 7-17 shows the general dimensions of, and the loads caused by, a coal-storage installation near the boiler house of an industrial plant in Indiana. It also contains the log of a boring taken at the center of the proposed construction. A foundation is to be designed for these silos.

Notice that a conveyor trench and a pit for the bucket elevator are to go between the silos. The bottom of each silo is to be a concrete slab supported upon a sloping earth fill in order to avoid the fire hazard from dead storage of combustible coal below the drawoff position. The silos must be able to withstand a heavy gale when empty or full.

The main clay stratum is rather soft. Since the silos may be almost fully loaded for long periods, the allowable bearing on this clay was set at 2 tons per ft.^{2}, with 3 tons maximum unit edge pressure from overturning and vertical loads. This is conservative, but tilting and considerable settlement must be prevented. The watertable was not definitely determined. However, little trouble from water is expected since most of the water is surface runoff or held by the relatively impermeable clay.

One suggested solution is the use of a 3-ft. ring of concrete 5 ft. deep, centered under the silo walls, as pictured in Fig. 7-18. A second is the scheme shown in Fig. 7-19. A third design is that pictured in Fig. 7-20. Which should be used? After examining these three proposals, the following comments should be studied:

Figure 7-18. 1. The weight of the silo would rest upon the wall. What would hold the weight of the coal in it? Generally, this would be trans-

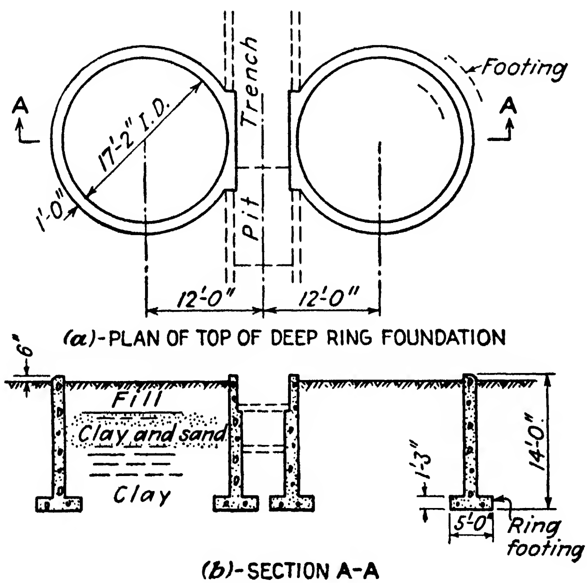


FIG. 7-19. Study of deep ring foundation for two coal silos.

mitted through the floor and fill to the ground and to the top of the inner portion of the ring. However, if the coal tends to arch and hang up during drawoff, much weight may be transmitted by friction into the structure and thence to the ring. The latter should be able to support a large part of the total weight if necessary. This would cause a larger soil pressure than allowable.

2. The ring walls have to be notched out at the conveyor trench and pit, causing local weakness.

3. The pit is deeper than the ring wall. This is not desirable.

4. If the trench-and-pit construction is not isolated from the rings, the former will tend to transmit unintended loads to the soil, and it may break off or unbalance the resistance.

5. There is a question as to whether the permanent dead load is sufficient to prevent the empty silos from tipping over in the wind.

6. On the whole, this foundation seems to be inadequate and undesirable

Figure 7-19. 1. The idea in this design is to make the rings thin but deep, thus clearing the trench and pit, engaging considerable earth to resist uplift, and transmitting the load to deeper stronger soil. The last does not appear to be accomplished because of the plastic nature of the clay. The deep overburden is not so beneficial with such material as it would be with sand.

2. The excavation and construction are considerably more costly. There is no necessity for placing the footings at this depth.

3. In general, the distribution of the pressure on the underlying clay will be somewhat like that shown in Sketch 7-18(c). Compaction of the deeper clay is therefore likely to cause the silos to tilt toward each other—something that should not be permitted.

4. The pressure from the contents of the silos may tend to load the top of the inward projection of the footings and cause a twisting tendency because this weight is offset from the reaction of the soil under the footing at any given section.

Figure 7-20. 1. In this design, the bottom of the elevator pit is made the position of the top of a continuous mat extending under both silos as well

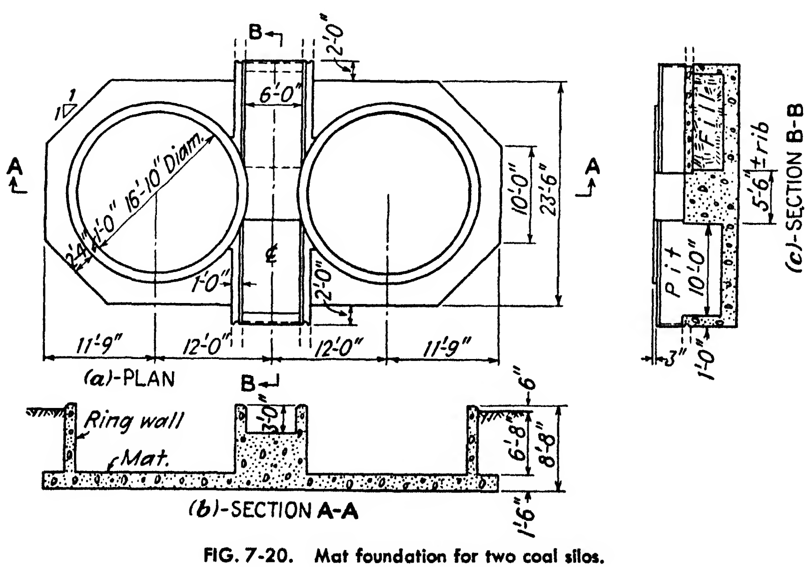


FIG. 7-20. Mat foundation for two coal silos.

as under the trench and pit. Whether the load is applied mostly through the fill under the drawoff floor, through the silo walls, or through both in varying amounts, the mat will distribute it over a large area at the top of the thick stratum of clay. As this area is considerably larger than that of the silos, the intensity of pressure is low.

2. The corners of the mat are nipped off, and the central portion is deliberately enlarged, in order to cause a tendency to produce tension in the top of the mat. This may counteract somewhat the tendency to produce tension in the bottom of the mat that will be caused by larger settlement of deep portions of the clay under the central region. Since the top of the clay has a rather low unit pressure, the lower layers will have still less, and the probable settlement will be smaller than for the other designs.

3. The trench and pit walls and the cross rib are used as stiffeners for the mat. This foundation will tend to resist relative tilting of the silos.

4. The base and fill on it provide adequate weight to resist overturning of the silos and prevent large edge pressures.

5. This plan may require more concrete than the others, but its total cost is probably less than that of Fig. 7-19, and it is inherently much more suitable for the job.

6. This plan should now be analyzed to see that it meets all requirements and to determine the reinforcement.

PROBLEMS

7-1. Design a waterproofed continuous mat as a basement floor for a building similar to that pictured in Fig. 7-1 for the following conditions: The soil is soft clay; the basement floor is 6 ft. below the ground and 9 ft. below the first floor; the bays are 20 ft. square; $P_1 = P_5 = P_6 = P_7 = 300$ kips, $P_2 = P_4 = 400$ kips, $P_3 = 450$ kips, and the corner column = 250 kips; no ribs or walls are to project inside the outer foundation walls.

7-2. Design a waterproofed mat for the conditions of Prob. 7-1 except for the addition of reinforced-concrete walls along the column lines in one direction only. Assume a 4-ft. doorway in the center of alternate bays along each wall, the center of the first door being 10 ft. from the outer foundation wall. Is this arrangement as economical and as safe as that designed for Prob. 7-1?

7-3. Design a ribbed-mat foundation similar in type to that of Fig. 7-9 for the following conditions: There are five rows of columns 18 ft. on centers in one direction and six rows of columns 20 ft. on centers at 90° thereto; the soil is medium silt for 10 ft. below the surface, 18 ft. of soft clay below that, then 12 ft. more of silt underlain by deep fine sand; there is to be no basement; the floor is to be 2 ft. above the adjacent ground; the assumed loads on each of the four corner columns = 200 kips; the other outer columns = 250 kips each; all interior columns = 350 kips apiece; the "cells" are to be backfilled with compacted gravel; an 8-in. reinforced-concrete floor is to cover all the space inside the outer 12-in. brick walls. What is the increase of unit pressure applied to the silt? Assume that the location is near Norfolk, Va.

7-4. Figure 7-21(a) shows the plan of a pit or sump 18 ft. deep that is to be built below the floor of a new industrial building. The soil profile is given in (b). The column foundations are not shown because they are at least 25 ft. from the pit, and

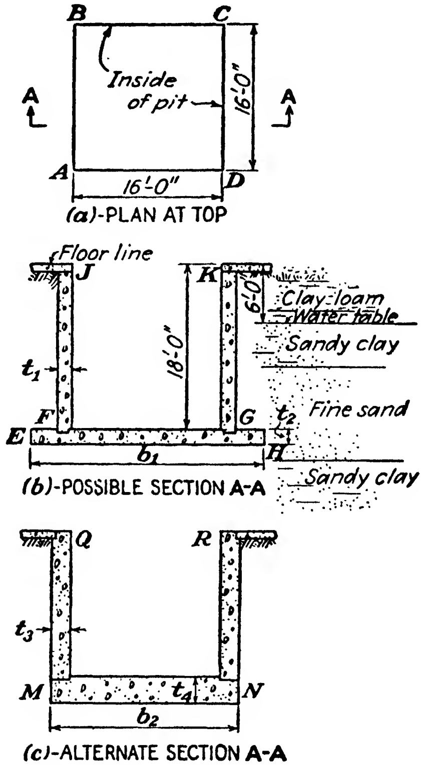


FIG. 7-21. Design for a deep pit.

therefore will not complicate the problem. Develop a plan for this pit, and show how it may be constructed.

Suggestions: One possible type of structure is pictured in (b); another, in (c). (1) Make a design that will resist buoyancy when the watertable is as shown; (2) analyze the sides and bottom, and plan the reinforcement; (3) plan the details of the membrane waterproofing; (4) plan the construction methods. If steel sheet piling is driven around the excavation and well into the sandy clay, will the latter prevent "blowing up" of the bottom when the excavation is unwatered?

7-5. The settling tank pictured in Fig. 7-22 is one of three proposed for the conditions and in the position shown; the one sump will serve all three tanks. The tanks are to be on an extensive, low area near a river, and the flood waters have been known to reach El. 101.5. Furthermore, any tank may be empty at the time of high water. Design the structures for the climatic conditions at Buffalo, N.Y.

Suggestions: Some details of the construction or such a tank are shown in Fig. 7-23. Can the buoyancy problem be overcome economically? Is it advisable to increase the fill over the general area in order to raise the tanks to a higher elevation? Is it safe to raise a tank so that the top of the footing of the ring wall is level with

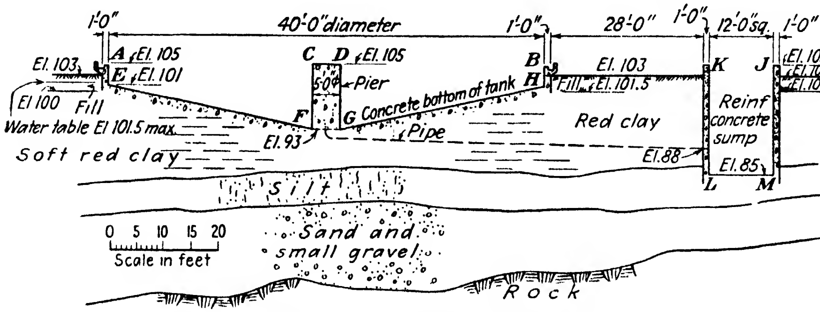


FIG. 7-22. Soil conditions at site of a settling tank and sump.

the ground? Will frost ruin a tank if the latter is emptied during winter weather? Can an automatic check valve or other flooding device be depended upon in case of floods at any and all seasons? If the bottom of the sump is set upon the rock, will

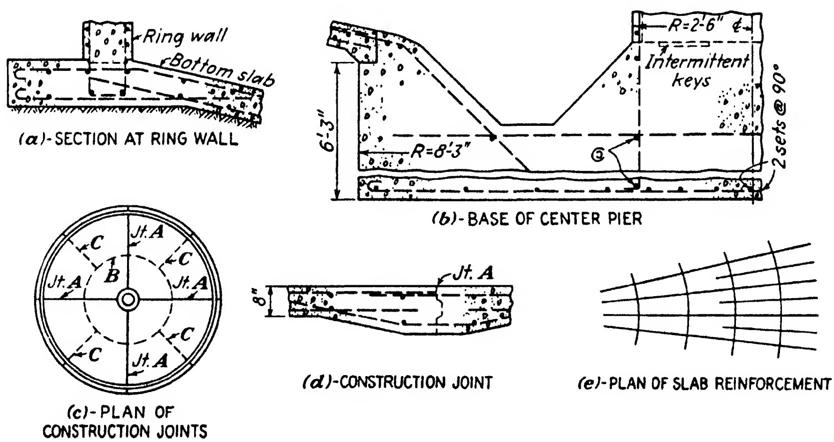


FIG. 7-23. Some details of construction of settling tank.

this eliminate buoyancy acting upon the sump? Do the tanks have to be waterproofed? Does the sump have to be waterproofed when the pumps and motors are supported upon its bottom? A good answer to each of these questions may suggest a solution for the problem.

8

FOUNDATIONS SUBJECTED TO OVERTURNING FORCES

8-1. Introduction. Most foundations are subjected to lateral forces in addition to vertical ones. In some cases, these lateral forces are important and should be considered in the design; in others, they may properly be "absorbed" by the safety factor. In no case, however, should they be disregarded as unimportant until the designer has satisfied himself that this may be done safely.

This chapter deals with some typical problems that are frequently encountered in the design of ordinary foundations. The principles illustrated are applicable to a wide range of engineering work; they will assist the reader to make sure that the foundations he designs will not slide, tip over, or be pulled out of the ground.

Critical conditions may be caused by wind on tall narrow light structures; by lateral loads from cranes and rapidly moving vehicles; by the unbalanced pull of cables and such equipment as belt conveyors; by pressures produced by earth, other granular materials, and water; by large eccentric loads; and by earthquakes.

The foundations discussed in this chapter are supposed to have footings or bases that rest directly upon the ground. Pile foundations are considered in Chap. 10.

The overturning effect caused by earthquakes is difficult to predict with any certainty. Local building codes (as for Los Angeles, Calif.) may specify the magnitude of the horizontal force to assume. This is usually expressed as some portion of the weight of the structure, such as 0.1 or 0.2, and it is often stated as an acceleration of $0.1g$ or $0.2g$. This force is assumed to be applied at the center of gravity of the mass of the structure. In general, structures with foundations on rock are less seriously endangered by earthquakes than are those that rest on deep granular or alluvial deposits which seem to magnify the vibrations somewhat as would a gelatinous mass.

For the purpose of analysis, overturning is considered in this chapter without giving special consideration as to how the magnitude of the forces and moments are obtained.

8-2. Connections at bases of steel columns. Consider the action of the base of a steel column that is subjected to an overturning moment. Some details for such steelwork are pictured in Fig. 8-1. The following comments refer to the correspondingly lettered sketches:

(*a*) This shows the assumed condition greatly simplified. This situation might occur in the case of a side-wall column of a tall mill building in which the frame action of each bent (columns and trusses in one row across the building) is to resist the wind pressure and other lateral forces and in which the bottoms of the columns are to be strongly restrained. The vertical load P and the bending moment M are to be resisted by the foundation, but the first requirement is the provision of sufficient strength and stiffness in the steelwork to transmit these to the concrete. The column shaft must be restrained directly, or indirectly through the base plate. It is customary to depend upon anchor bolts as the direct connection for resisting any tension caused by overturning. Columns may be embedded in the concrete, but this is seldom effective in resisting bending unless the embedment is 3 or 4 ft. deep, and the concrete is designed to provide the necessary reactions.

(*b*) A milled column shaft resting directly and squarely upon a base plate will offer some resistance to rotation because of the "riding" of the leeward flange B as shown here. The greatest resistance, however, is limited to the moment $Pd/2$, in which P should be assumed equal to its minimum value—the dead load only. This resisting moment is usually too small and too questionable to be trusted completely unless P is very large compared to M.

(*c*) Here base angles are connected to the column and the anchor bolts, the base plate being for the distribution of pressures only. Unless angle C is very stiff, it will bend as shown to exaggerated scale, it will yield excessively, and its distortion may cause the top of the anchor bolts to bend or break. It is obvious that these bolts must provide any necessary downward reaction T, and that the pressure $P + T$ tends to be concentrated near the leeward flange. It is desirable to make the connections strong enough so that M can be resisted by the moment TL unless the moment $Pd/2$ is relatively large and trustworthy, in which case one may assume $M = TL + Pd/2$. However, the magnitude of the wind pressure is unknown. It may be assumed to equal 20 p.s.f. acting upon the vertical projection of buildings under 50 ft. high; 30 p.s.f. on taller structures; and sometimes 40 p.s.f. in regions subject to hurricanes and cyclones. The local effect of gusts is anybody's guess. In any case, one can easily imagine the concentration of pressure in the steelwork and masonry in the vicinity of D, and this pressure should be provided for.

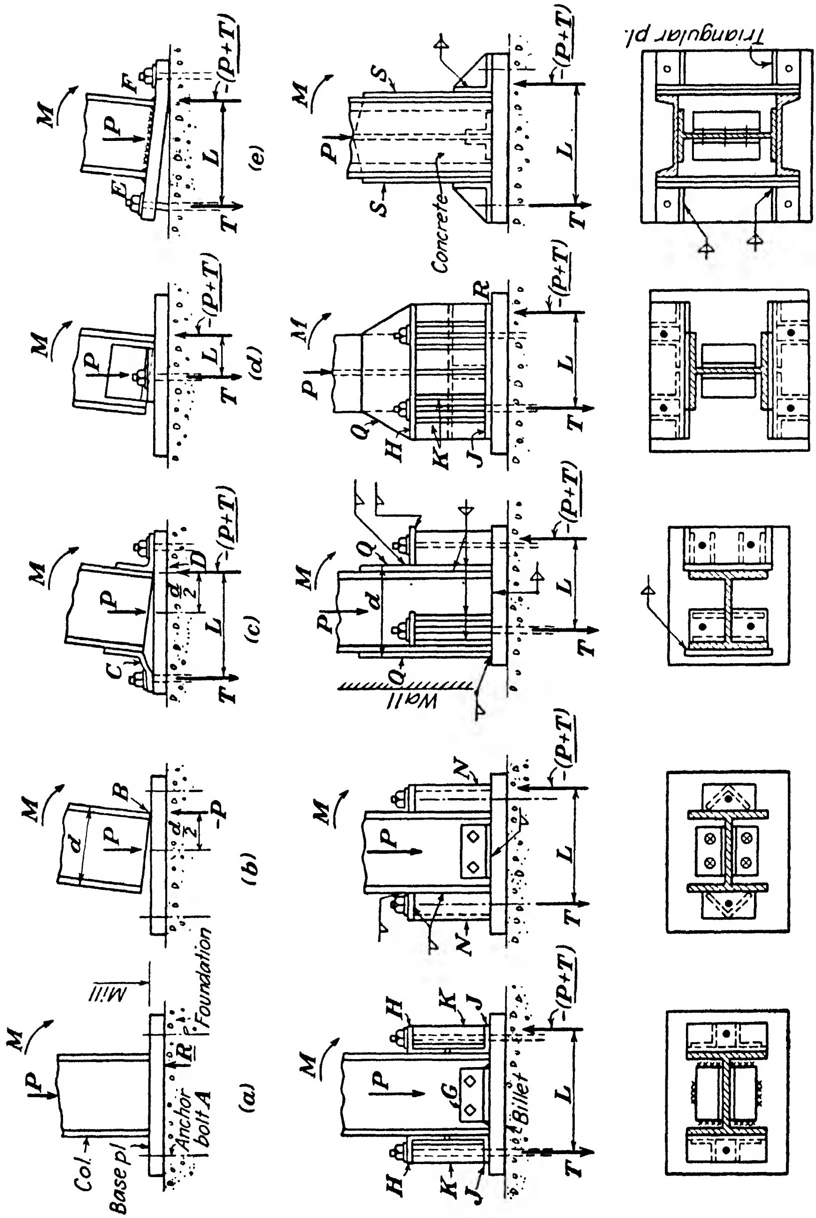


FIG. 8-1. Connections of steel columns to concrete foundations to resist overturning.

(*d*) This sketch shows the ineffectiveness of web angles on the column, because of both the short lever arm L and the warping of the outstanding legs of the angles. When the wind is at 90° to the web of the column, these angles are also relatively weak.

(*e*) This pictures a base plate welded to the column shaft. If the plate is thick enough, if the welds are adequate, and if the concrete can resist safely, this arrangement may provide a greater lever arm L and a more effective resistance. However, this involves field welding or difficulties in setting the column, as will be explained in connection with grouting.

(*f*) This illustration shows one arrangement for "boots" at a column base to prevent weakness caused by the yielding of details. Angles G are connected by rivets or welds to the billet and do little except transmit shearing forces when necessary. The base angles J should be milled with the column shaft after fabrication, or be set very carefully and made flush with the milled end of the column. Angles H serve as a shelf for the washers and nuts of the anchor bolts, but their resistance depends mostly upon the stiffeners K. Not only may L be increased in this manner, but reliability and stiffness are secured. However, if P and M are large, there may be an unsafe concentration of pressure at the outer tips of angles K on the leeward side.

(*g*) This is a minor modification of Sketch (*f*). It is arranged for welded construction and to avoid thin outstanding material at the point of maximum compression at the leeward edge. The welded connection of angles N or the compressive stress at the corner of these parts may then be the critical feature. All the horizontal shear must be resisted by the web angles and friction because the anchor bolts stand free of angles N. Of course, the bolts and steelwork inside of these angles cannot be painted after erection.

(*h*) Here the column is set so close to the face of the exterior wall that one pair of anchor bolts is placed inside the outer flange. It is obvious that the lever arm of the anchor bolts for resistance against tipping is probably less when the overturning is as shown than when the wind acts in the opposite direction. The flange plates Q may be added when more metal is needed in bearing than is available in the column flange alone, especially when the column tends to rotate about the tips of the flanges, 90° to the direction shown here. When d is large and when obstructions outside of the column are objectionable, both sets of anchor bolts may be placed inside the flanges.

(*i*) This pictures a case where the wind is acting perpendicular to the column web. In some cases, it may be desirable to flare the reinforcing plates Q to provide a greater lever arm L. The angles J should be extended, too, and both J and Q should be thick and milled to bear. Even so, the edge R is likely to yield under the concentrated compression if the anchor bolts are stretched appreciably. Sometimes, as for cantilevered poles supporting electric lines, the entire base may be widened so that Q, H, J, and K may accommodate three or four anchor bolts in a row.

(j) Here a heavy column is shown with channel cover plates and stiffening plates S so that large overturning moments may be resisted in any direction. Plates S are to be thick enough to enable the compression to act safely at the edge, and the channel flanges should enable the section to resist local buckling. This sketch shows triangular welded plates to stiffen the base angles. These plates are helpful but are not a complete substitute for the much stronger "boots."

8-3. Anchor bolts. If an anchor bolt is to resist tension, its connection to the concrete must be adequate. A plain straight rod embedded in concrete may not develop adequate bond to prevent the bolt from pulling out, or even from twisting when one tries to loosen a badly rusted nut. When an anchor bolt is not completely embedded in concrete, the exposed end may corrode badly, hence the critical section at the root of the thread should be designed so that the computed maximum tension is only 12,000 to 16,000 p.s.i. A strong, plate washer is generally desirable under the nut; and double nuts are sometimes used when it is advisable to "lock" them in place. Shouldered bolts are used occasionally when tension is to be resisted, but the connection must be loose enough to permit a slight lateral movement of the connected part. One should not rely upon a workman's "tightening a nut lightly" so that some shearing movement can take place; he will generally draw it up tightly. Hexagonal nuts are preferable to square ones because they can be engaged by a wrench more easily in narrow spaces.

Referring to Fig. 8-2, the following comments are given as a general guide:

(a) This bolt is "hooked" to increase its anchorage and to prevent turning. The projection should be 4 to 6 diameters. The embedment L should preferably be at least 40 diameters, or more for machinery subject to sudden or repeated tensile loads.

(b) Plain rods are almost necessarily used when making anchor bolts. However, such a rod may be deformed cold or when heated to make a swedged bolt. Such a type is useful when anchor bolts are to be grouted into holes drilled in rock or concrete. Because of the danger of trapped air pockets that prevent thorough grouting, it is often desirable to fill the hole with grout first, then to force the anchor bolt into position. Lead, or iron filings mixed with sulphur, may be used in lieu of portland-cement grout. Another danger to be guarded against in such drilled anchorages is the presence of seams or cracks that limit the resistance of the anchor bolt to the weight of a relatively small volume of rock. The diameter of the drilled hole should be at least 2½ in. or 3 to 4 diameters, and its depth should be 40 to 60 diameters.

(c) This hacked bolt is for service similar to that shown in (b). However, the former is generally easier to manufacture.

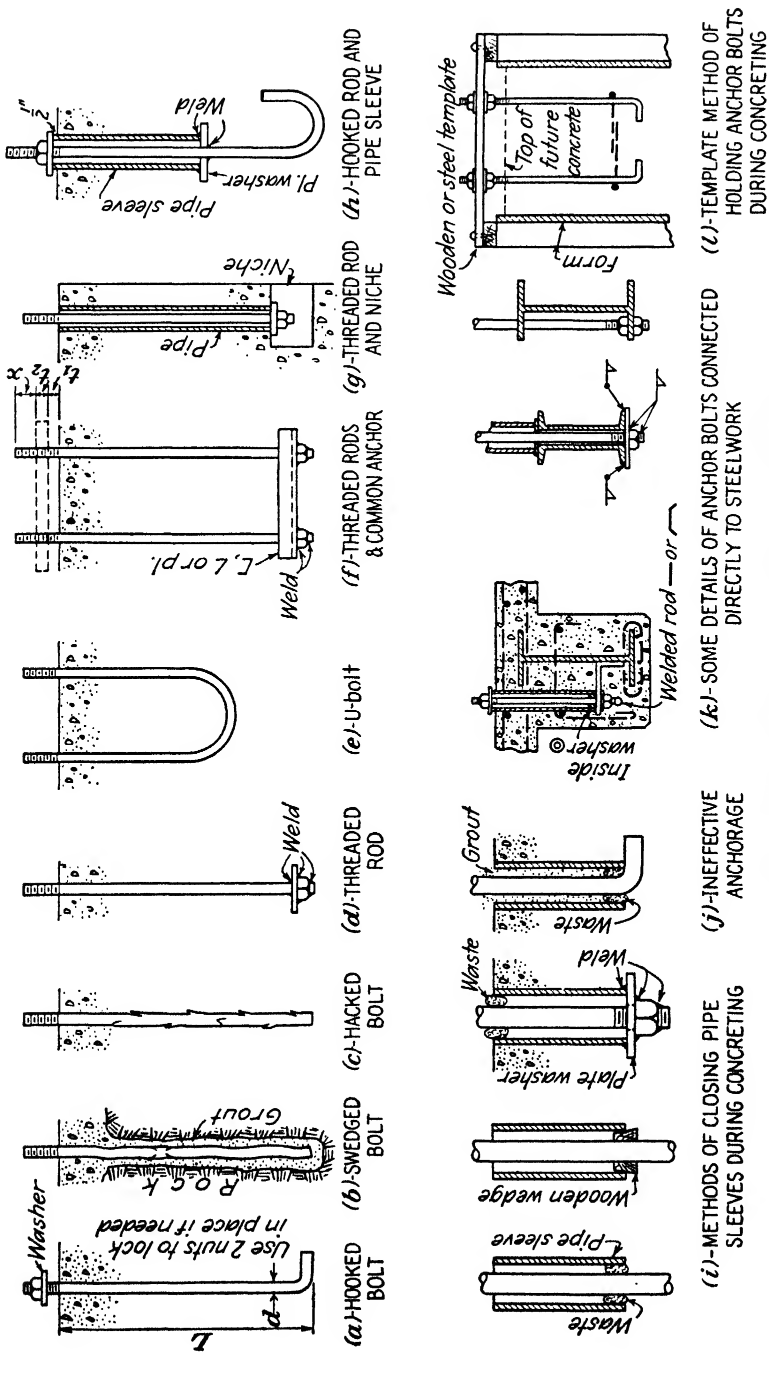


FIG. 8-2. Some types and details of anchor bolts and pipe sleeves.

(*d*) Seldom are anchor bolts so small and short that it is feasible to use regular formed heads on them. This drawing shows the substitution of a rod threaded at both ends. The washer is to resist pulling out, the nut is to bear against the washer, and the welding is to prevent turning of the rod. This bolt is generally more costly than (*a*).

(*e*) A threaded rod bent into a U may seem to be desirable when two bolts are to be near together. It is difficult, however, to fabricate it exactly right, and deep embedment is still needed if the concrete is to develop sufficient tensile resistance.

(*f*) Two rods may be attached to a large common anchor as shown. Welding is desirable here as in (*d*). In all cases, the threaded length and the projection above the concrete should be adequate. Here t_1 is the allowance for grouting; t_2, for the connected steelwork; and x, for washers and nuts plus some leeway.

(*g*) In the case of machines, the anchor bolts may have to be removable. Here a threaded rod is extended through a pipe sleeve to a niche where a washer and nut can be attached. When machines are anchored to floors or beams, the pipes may go clear through the concrete so that the rods may be inserted from above or below. If this is not done, be sure that the bolts can be inserted from above after erection of the machine. Another type of removable anchor bolt is one with a short head similar to a hammer. This may be placed through a slot in an embedded insert, then rotated 90°, and tightened to hold it in position.

(*h*) There is an old saying that "if the anchor bolts are not set correctly, the structure will not fit." This sketch shows a rod with a 180° hook—seldom necessary—and a pipe sleeve. The latter is to permit a moderate amount of springing of the bolt to fit the holes in the attached base. The washer is to close the bottom of the pipe against the entrance of wet concrete during pouring; the welding is to hold the washer and sleeve in proper position; the ½-in. projection is to raise the pipe sleeve above the level of the poured concrete but it should be considerably less than the allowance for grouting. The washer and nut may be used temporarily to assist in keeping out the concrete. The effective anchorage of the bolt is only the part below the pipe sleeve. Cardboard tubing and wrappings of tar paper are generally poor substitutes for pipe sleeves, except in such a case as that in (*g*).

(*i*) This shows other ways of closing the ends of pipe sleeves. Waste is obviously weak and unreliable, and wooden washers may drop off when the concrete is compacted. The third illustration shows an arrangement for anchor bolts that must necessarily be relatively short. Here the large plate washer must support the pull in the bolt except for whatever bond resistance may be developed on the outside of the pipe. If the nut is not welded to the rod, this bolt may be unscrewed, removed, and then replaced. However, if the threads become damaged, these operations cannot be continued, and remedial measures are difficult.

(*j*) This sketch pictures the absurd results that may be obtained if one pursues too far the philosophy, "if a little of something is good, more of it is still better." To be specific, long pipe sleeves permit one to spring an anchor bolt sidewise more easily than do short sleeves. However, when the depth of embedment is small and limited, the anchorage of the bolt itself may be ineffective. As shown here, the hook cannot develop the proper anchorage because its strength as a cantilever is relatively small, and the length is too little to develop appreciable bond resistance.

(*k*) These drawings show a few details for use when the anchor bolts for machines are attached directly to steelwork. In the first illustration, field welding is avoided by inserting the threaded rod upward through the hole in the shelf angle, then dropping the pipe sleeve and the centralizing washer over the bolt, and finally tightening the nut to hold the assembly during the concreting. The cross rod at the bottom is to prevent twisting of the bolt when the machinery is erected or removed. The nut and wide washer at the bottom of the second sketch should be shop-welded to the bolt, then the washer may be field-welded on top because bottom welding is more difficult. When the steel is bare, some physical connection is needed to hold up the bolt except when it is supposed to be removable; it may drop out or be knocked out when the machine or steelwork is placed over its top. In the third sketch, the two nuts are supposed to lock the bolt in position. This principle may be used with various modifications when the bolt must be erected from above.

(*l*) It is generally essential to use some sort of template of wood or steel to hold anchor bolts in the correct positions vertically and horizontally during the placing of concrete. This sketch illustrates a simple method for supporting the anchor bolts in a pedestal that is to be under a steel column. When planning the structural details, one should endeavor to arrange a group of four anchor bolts so that they are spaced equally in both directions or so that the inequality is obvious. For example, if the spacing in one direction is ½ in. less than in the other, the erectors may easily make the mistake of rotating the group 90° from the position intended. Furthermore, varieties of pattern and spacing should be minimized. There have been cases in which the wrong group of bolts was put in a particular place, necessitating chipping of concrete, bending or burning of bolts, welding of offset extensions, drilling of holes and insertion of expansion bolts, or even the omission of some or all of the intended connection to anchor bolts. In any case, templates for erection should be strong, easily placed, well supported, undisturbed by concreting, and such that the bolts can be located properly by the surveyors or others. If one can be sure that the anchor bolts are set accurately, the need for troublesome pipe sleeves will be eliminated because the shopwork done on the metallic parts when drilling the holes in them is generally trustworthy. The pipe sleeves are to allow for inaccurate fieldwork.

Figure 8-3 shows the general conditions prevailing in one case where the anchor bolts at the head end of a heavy metallic conveyor pulled out. The tractive force caused an upward pull on bolts A; and the first time that the machine was operated under full load, these bolts gave way. Holes were

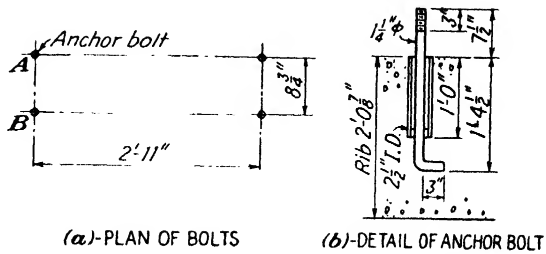


FIG. 8-3. Details of anchor bolts that pulled out.

then drilled through the reinforced-concrete beams under the bearings, and through bolts were installed—all at considerable expense and loss of production.

Another unfortunate case is pictured in Fig. 8-4. This illustrates the trouble that may result from a "little" knowledge. A 5-ft. cylindrical pier was to support the machinery to operate the rotating equipment in a 100-ft.-diameter settling tank. The equipment was to be attached to the top of the ring of concrete. The designer realized that a construction joint at A

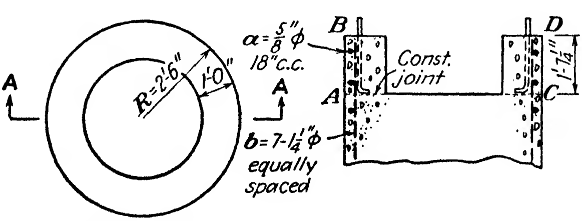


FIG. 8-4. A pivot pier that failed because the anchor bolts were not embedded properly.

was essential. He thought that it would be economical to pour the main shaft and then place the anchor bolts on top of this concrete before pouring the ring. (This was contrary to specific orders.) When the machinery was started, the large horizontal torque sheared off the ring at A. The ends of the seven vertical rods b were hopelessly inadequate to "tie" the ring in place, and the circumferential rod a was, of course, merely a spacer. The final "economy" was the cost of complete dismantling of the equipment, cutting off 3 ft. 6 in. of the top of the pier, the erection of long anchor bolts, an increased amount of reinforcement, the pouring of more concrete, and a serious delay in starting production. Furthermore, the same work had to be done to a duplicate tank alongside the one that failed.

If the space around an anchor bolt in a pipe sleeve is left empty, water may enter and cause some corrosion and, if a structure is exposed to freezing, cracking of the concrete may result. It is desirable under such conditions to fill the sleeve with grout, as pictured in Fig. 8-2(j).

Anchor bolts that must resist large transverse shearing forces require special attention by the designer. In most ordinary structures, the vertical loads are likely to be so large that frictional resistance under pressure is sufficient to withstand all probable lateral forces. In the case of machines and some special structures, large transverse shears may be applied to the anchor bolts.

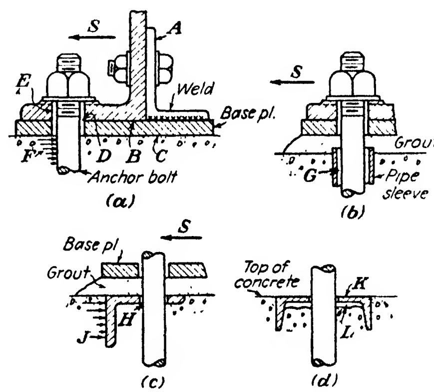


FIG. 8-5. Shearing resistance of anchor bolts.

As shown in Fig. 8-5(a), the holes in the steelwork are usually made ¼ to ½ in. larger than the anchor bolt. If the structure or frame is fastened to a base plate by some adequate connection A, the shear S will be transferred to this plate without depending upon the anchor bolt. Otherwise, slippage may occur either along surface B or along C until the space D is closed and the bolt offers resistance. However, this resistance is likely to be dependent upon the local crushing strength of the grout or concrete at F. When the bolt is in an empty pipe sleeve, the anchor bolt will not develop effective resistance until the slippage occurs and the space G in Sketch (b) is closed. Bearing against the threads of an anchor bolt will damage them, but this weakness is generally inconsequential. The pipe sleeve and spaces D and E, Sketch (a), may be filled with grout, sulphur and iron filings, lead, or any material that may be packed in and will ultimately have sufficient strength in bearing.

It is possible, in special cases, to use small angles like J in Sketch (c) with holes H having a clearance of only 1/16 in. The bolts may bear against the steel at H, and this angle will bear against a sufficient area of concrete. It is also possible to set the bolts in a permanent embedded frame. However, one should be sure that the frame is properly encased. For example, if a channel K with its flanges turned down is substituted for angle J, as shown in Sketch (d), the lower portion L may not be filled with concrete. Details utilizing the principles shown in Fig. 8-7(d) may also be employed to lock the steelwork and concrete together mechanically.

When pipe sleeves are used in a foundation, they tend to cause planes of weakness, especially if two or more are close together. This is especially serious in the case of vibrating machinery and when there are thrusts or shocks that cause the anchor bolts to bear upon the inside of the sleeves. The concrete around the anchor bolts should be well reinforced. It is desirable to tie the pipes in one row together, and to tie those on two opposite sides of a pedestal, by using "hairpin" rods looped around the sleeves. Three foundations for large machines cracked along the sleeves because this reinforcement was too light. Two rows of hoops around the top of a pedestal were insufficient; three or four rows with at least two rows of intermediate hairpins at the pipes probably would have been satisfactory.

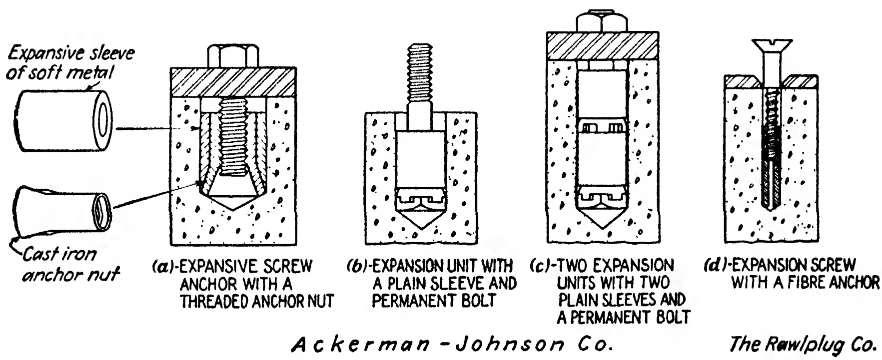


FIG. 8-6. A few types of expansion bolt.

Expansion bolts such as those pictured in Fig. 8-6 are useful when the attachments of machines or steelwork must be made to old concrete, and when the locations of anchor bolts cannot be ascertained before new concrete work is done. The following comments should be considered:

1. Expansion bolts may be able to resist a moderate tension if the concrete will withstand the pull; sometimes a questionable matter.

2. They are generally weak in resistance to transverse shear because their tops are not supported laterally unless well grouted.

3. Usually, the steelwork must be set, the holes located, the steel removed, the holes drilled, the bolts installed, and the steelwork reerected. There must be space for performing these operations.

4. The cost of labor for fieldwork may exceed that of setting standard anchor bolts.

5. Some types of expansion bolts may be removed—an asset in the case of machinery.

6. Drilling of holes and the wedging action of expansion bolts are likely to crack or spall narrow concrete members, such as curbing and the ring

shown in Fig. 8-4. The same results may occur if the bolts are near the edge of wide concrete sections.

7. If the drill strikes a reinforcing rod near the surface of the concrete, relocation of the bolt may be necessary. Therefore, the reinforcement should be planned to expedite the installation of the expansion bolts. Near edges or corners, the rods should be arranged to tie the concrete together. They may prevent serious spalling but are not likely to stop the cracking of the concrete.

8. Expansion bolts are generally small in size and should seldom be used when the tensile forces are large. Occasionally it may be possible to leave wedge-shaped pockets in large concrete bases of machines so that the equipment may be set, the bolts placed, and the pockets filled with concrete to lock the bolts in position.

It is obvious that reinforced-concrete columns or piers subject to overturning moments, as in Fig. 8-10, may and should be tied into their foundations by means of dowels or extensions of the main reinforcement. Great refinement of fieldwork, however, is seldom necessary.

8-4. Grouting. The use of portland-cement grout and mortar under bearings has already been referred to. One can easily realize that workmen should not be expected to finish the top of a concrete foundation to the exact desired elevation and to a true level plane surface. Wet shrinkage of concrete, troweling around anchor bolts, the formation of laitance, vibration of neighboring concrete, hasty surveys to determine elevation and true level—these make accurate concrete work difficult and almost unobtainable. For minor parts, it may be satisfactory to set steelwork directly upon a finished concrete surface with a thin intervening layer of dry cement or a paint coat of neat cement grout. It is *possible*—but inadvisable—to pour the concrete slightly high and then grind or bushhammer it down to the desired elevation. Accurate workmanship, however, is next to impossible amidst anchor bolts, and the cost is beyond all reasonableness.

A simple case of grouting is illustrated in Fig. 8-7. A large column generally requires a heavy base plate, as shown in (*a*). Such a billet cannot be placed by hand. Angles *A* are useful for the attachment of a chain or hook so that a crane can lift the plate into position over the anchor bolts. The bolts should be set with some predetermined allowance t_1 for *grouting* (this word being synonymous with *mortaring*). A few of the procedures and possibilities of grouting are shown by the following explanations of the sketches in Fig. 8-7:

(*a*) A portland-cement mortar (1:1½ or 1:2) might be spread over the top of the pedestal, then the base plate might be set and forced down to proper grade and level. Of course, this method is inadvisable because it is difficult to secure sufficient accuracy of erection. It is also apparent that the erection of a tall column with the base plate attached would be far more difficult because the column would have to be steadied laterally for a day or two, and the great weight would probably squeeze out the mortar to an unpredictable extent. In any case, some reliable control is needed.

(b) Here the base plate is supported upon steel wedges that permit proper placement of the billet. A form is built around the base, and a "wet" mortar is placed around and under the latter. There is likely to be an air pocket near the center so that the plate is not properly supported. It is possible to lift up the plate and fill any low places with mortar, then to

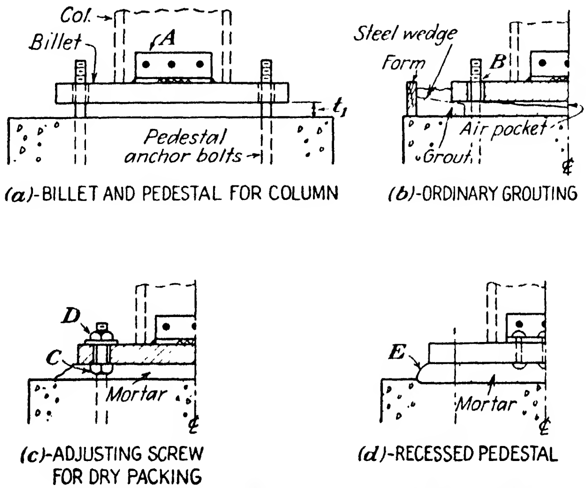


FIG. 8-7. Arrangements for grouting a heavy base.

replace the billet. This extra work costs money, and it may be difficult to secure proper contact with the wedges the second time. If the plate is adjusted and removed, the mortar placed, and the base reerected, similar difficulties ensue. After the mortar has set, the wedges should be removed and the spaces pointed up. Sometimes, one or two large holes are made near the center of a large base so that grout may be placed through them, or so that the elevation of the mortar may be observed and (it is hoped) the latter may be made to bear against the steel.

Special compounds are manufactured for use in grouting, either as an admixture or as a substitute for cement mortar and grout. Wet shrinkage and slumping of mortar may be troublesome; chemical shrinkage will probably be very small; and the quality of the workmanship may be more important than slight variations of materials.

(c) By threading the anchor bolts sufficiently, two nuts C and D may be used as adjusting screws. The base plate may be erected and positioned accurately, then a stiff, "dry" mortar may be packed under the base and rammed into place with a stick and mallet. Space for this work is necessary around the base. It is almost essential to have four anchor bolts in any base plate because two bolts may not prevent rotation of the base about the line between them when the mortar is packed on one side more than on the other. It is obvious that some pressure will be exerted on nut C by the structure, but this is inconsequential when the mortaring is done well. In this way, any sensible workman can obtain a trustworthy bearing. It is also possible, when this screw device is used, to remove the base, deposit

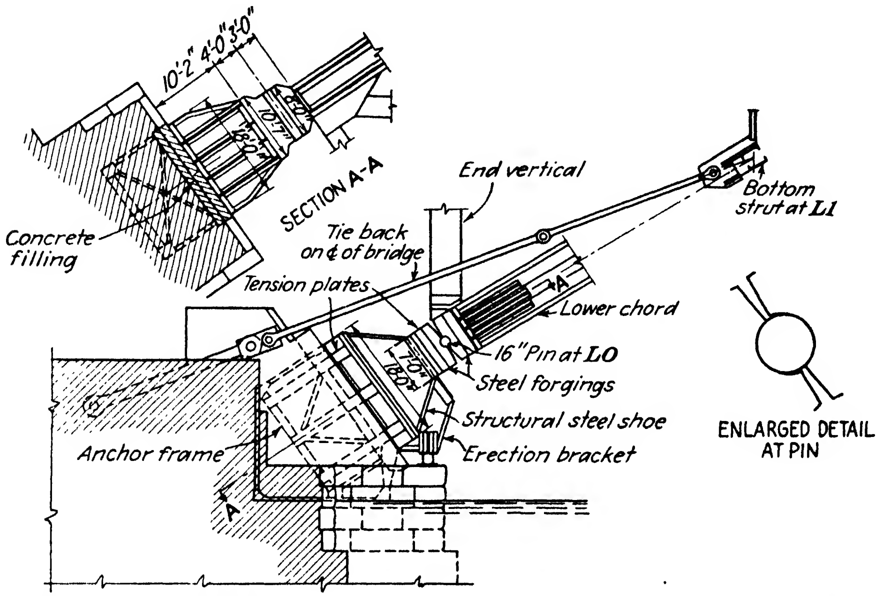


FIG. 8-8. Details of the bearings and erection frames of the Bayonne Bridge. *(Courtesy of the Port of New York Authority.)*

and smooth off the mortar, replace the billet, and tighten nuts D so as to force the base back into position, except for whatever mortar may remain between the plate and nuts C.

(*d*) Under very heavy loads, and especially under machinery subjected to shocks, it may be desirable to have means for clinching the mortar in place in case it cracks. This pedestal is recessed at E; the base may be dry-packed; but the mortar pad cannot become loose. A scored or roughened top finish of the pedestal might provide similar results. Wire mesh in the mortar may prevent the latter from falling apart, but it probably will not prevent the cracks in the first place.

The customary allowance for grouting is 1 to 2 in.; less than that is difficult to place. Under very large machines, 3- or 4-in. thicknesses may be desirable. In the construction of the bearings of the Bayonne Bridge, the main structural shoes were riveted to a steel frame, and then a 2-ft.

layer of reinforced concrete was placed below the shoes, as indicated in Fig. 8-8.

8-5. Action of steel base plates. The minimum area of a steel base plate under a column or under a bridge bearing is generally determined by dividing the computed load by 600, 800, or 1,000 p.s.i., or by some other specified or selected allowable unit stress on the concrete. The base plate is usually relatively thin, made of a ductile material, and designed for a large unit stress in bending, and the load is generally applied to it at a few points or lines of large concentration. It is therefore probable that a base plate under a column, for example, cannot spread the concentrated load uniformly over the concrete and mortar or grout directly under the steel. It seems reasonable to believe that the plate will deform into a saucerlike shape, the edges curling upward. This would tend to cause a high unit pressure on the concrete under the shaft of the column, with decreasing intensities toward the edges of the plate.

When there is a large bending moment that tries to rotate the column, as pictured in Fig. 8-9(a), the column tends to bear heavily on one side and cause a very highly concentrated load at some line such as F. It is probable that the base plate will tend to deform, as pictured in exaggerated manner in (b). At least, it is evident that the resultant of the compressive stresses in the concrete must counteract and line up with the applied load P, and this indicates the probability of large stresses in the vicinity of G. The

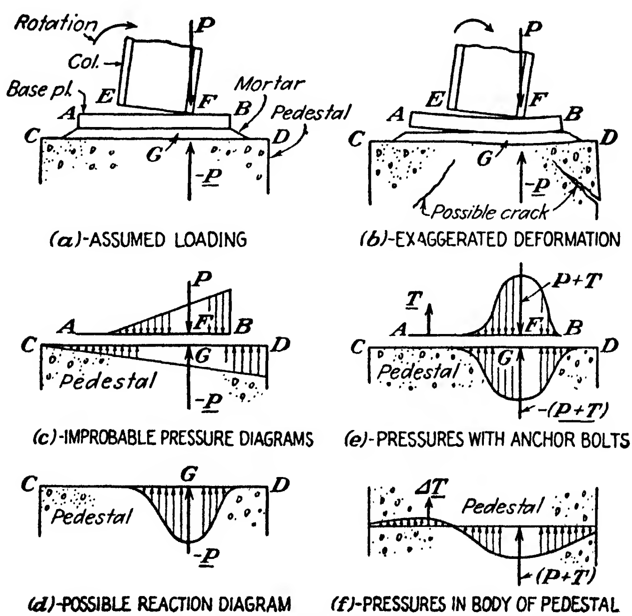


FIG. 8-9. Illustration of possible action of a steel base plate and a concrete pedestal.

edges A and B may not lift up as pictured, but the former particularly will not apply much if any pressure. It is obvious that the large compressive stresses and the deformation near G will tend to cause cracking of the corner D, and possibly of C.

This exaggerated picture in Fig. 8-9(*b*) shows why it is desirable to use a conservative value for the design stress under a base plate, why the area of the top of the pedestal should be large so that weak corners are not near points of large compression, and why the tops of pedestals should be tied across and around with reinforcement to prevent dangerous and visible cracking. The sketch shows also that the probable pressure diagram under the base plate is more like that pictured in (*d*) than the one in (*c*). In general, a large area of thick concrete well tied together will withstand a very high localized and centrally applied pressure, but most practicable concrete pedestals and bases are not large enough to trust this action. Large loads causing high localized pressures near edges and upon narrow concrete sections should have the foundation concrete proportioned conservatively.

If the overturning moment is sufficient to cause tension in the anchor bolts, as pictured in Fig. 8-9(*e*), the total resultant pressure under the base plate and on the concrete is increased by the magnitude of T. Down in the body of the pedestal, the pressure diagram may be somewhat like that in (*f*), where ΔT represents local tensile stresses caused by the bond of the concrete to the anchor bolts. The compressive stresses in the pedestal are generally unimportant if the top is strong enough.

8-6. Isolated footings subjected to overturning. In the design of practicable structures there are many instances in which isolated footings are subjected to vertical and overturning forces. When the latter are small compared to the former, the overturning may be neglected, its effect being assumed as taken care of by the safety factor. However, overturning should be investigated to see whether or not it is important.

When the maximum bearing value of the soil is known with reasonable certainty and when a safety factor of 1.5 to 2.0 has been used in establishing the allowable unit bearing pressure p for design purposes, one may properly ask whether p may be increased by some percentage when designing foundations for dead, live, and lateral loads all acting simultaneously. It would seem that the improbability of the absolute maximum loads and the temporary nature of these combined loads justify some increase in the allowable bearing pressure. The author suggests the following specifications, where p is the allowable bearing value for direct loads:

For dead load plus live load.	p
For dead load plus wind and other lateral loads.	p
For dead load plus live load plus wind and other lateral loads.	$1.2–1.3p$

As a simple case, assume the footing pictured in Fig. 8-10(a), with the loads as shown. The vertical load P will be assumed to equal the weight of the superstructure, its loads, and the weight of the complete substructure, including the earth above the footing GH. Let M represent the overturning moment without consideration of any accompanying horizontal forces. Probably the intensity of the pressure under GH will not be distributed uniformly, nor will it vary as a straight line. Nevertheless, it is safe to make these assumptions. Therefore, there may be two varieties of pressure diagram:

In the first, using L for GH and b for the width of the footing at 90° thereto, the uniform pressure caused by P is

$$p = \frac{P}{Lb} \tag{8-1}$$

The pressures p' supposedly caused by the overturning moment will be, for a rectangular footing,

$$p' = \frac{Mc}{I} = \frac{6M}{bL^2} \tag{8-2}$$

When p' is less than p, the combined pressure diagram is represented by Fig. 8-10(e). If the eccentricity of the resultant pressure at the base is

$$e = \frac{M}{P} \tag{8-3}$$

then

$$\left.\begin{aligned} p_1 &= \frac{P}{bL}\left(1 + \frac{6e}{L}\right) \\ p_2 &= \frac{P}{bL}\left(1 - \frac{6e}{L}\right) \end{aligned}\right\} \tag{8-4}$$

The second case occurs when p', of Sketch (d), exceeds p, the eccentricity exceeds $\frac{1}{6}L$, and the resultant load hits outside of the middle third of the footing GH. Since tensile resistance of soil sticking to the footing near J of (b) cannot be relied upon, it is satisfactory to assume a triangular distribution of pressure, as illustrated in (f). The maximum pressure at K is computed as follows:

$$e = \frac{M}{P} \text{ (as before)}$$

This locates the position of the resultant reaction $-P$, which will be at the center of gravity of the pressure diagram. Therefore,

$$\frac{L}{2} - e = \frac{L_1}{3} \qquad \text{and} \qquad L_1 = 3\left(\frac{L}{2} - e\right) \tag{8-5}$$

$$\tfrac{1}{2}p_1L_1b = P \qquad \text{or} \qquad p_1 = \frac{2P}{3b[(L/2) - e]} \tag{8-6}$$

In connection with Figs. 8-10(a) and (b), notice the following details in the respective sketches:

(a) In any case, dowels A should be large enough and long enough to splice rods B and to distribute their stresses into the massive concrete. If rods B are in tension, it is advisable to hook the dowels at the bottom to engage the main footing; these hooks are of little value in resisting downward compressions. The construction joints at C and D are roughened,

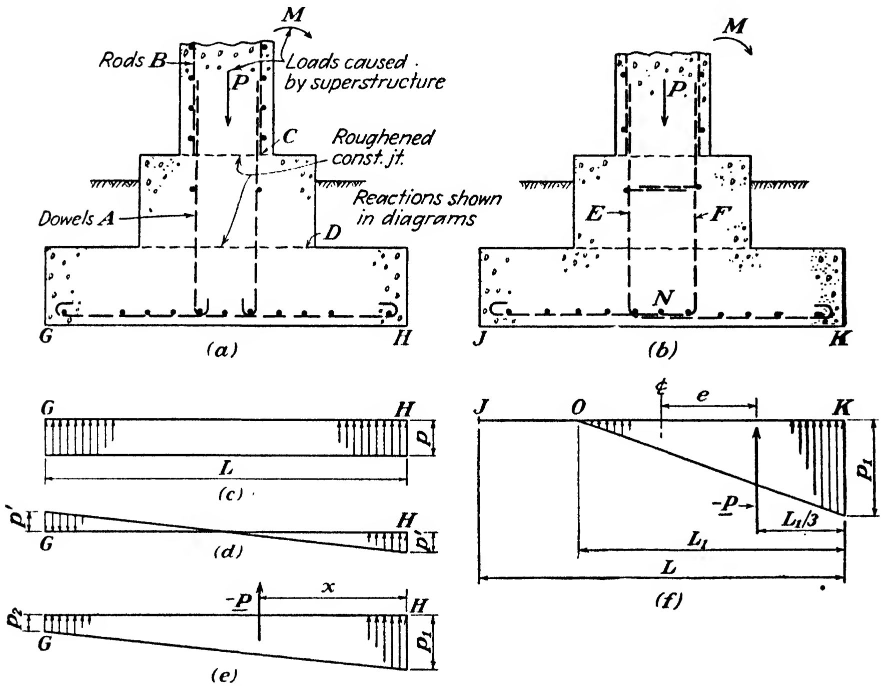


FIG. 8-10. Assumed pressures under an isolated concrete footing with direct and overturning forces.

but no actual keys are necessary because of the large pressure and friction caused by P.

(b) Dowels E and F are shown bent toward K and J, respectively. This may seem to be efficient, but it is usually undesirable because of questionable bond stresses in the vicinity of N, and regular hooked rods are needed alongside E and F anyway in both directions. A long U-rod from E, across N, and up F is better.

The maximum pressures p_1 in Figs. 8-10(e) and (f) are gross intensities. The weight of the displaced earth and backfill above H and K may be deducted from p_1 if one wishes to consider the increase of pressure only. In such problems as these, this refinement is not generally justified as far as pressure on the soil is concerned, but it may properly be considered when proportioning the concrete and reinforcement of the substructure.

Figure 8-11 shows some types of footing that may be used. In these cases, the overturning is assumed to be caused by the shear S that may be the result of frame action, wind, or bracing connected to a steel column. The eccentricity e may be found as follows, using h for the total height of the footing:

$$\frac{e}{h} = \frac{S}{P} \quad \text{or} \quad e = \frac{Sh}{P} \tag{8-7}$$

The load P equals the weight on the column plus that of the foundation concrete and the earth above the footing. In the case of (a), the effectiveness of the earth around the concrete and above C and D may be questionable because of the wedging action of the trapezoidal shape.

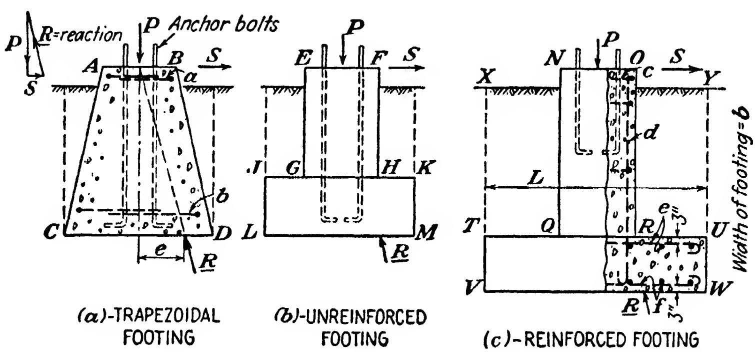


FIG. 8-11. Isolated footings for resisting small overturning forces.

In the sketches in Fig. 8-11, notice the following:

(a) The anchor bolts are extended close to the bottom of the footing so as to engage the entire weight of the footing without the use of vertical reinforcement. The only rods that may be of appreciable value are the hoops a and b to tie in the top and bottom of the footing; even these are a sort of insurance rather than a necessity. The resistance of this footing to overturning is generally small because the base CD is usually narrow.

(b) The anchor bolts in this unreinforced footing are also extended close to the bottom. They reinforce the pedestal automatically. Both P and S are supposedly small, otherwise there may be danger of cracking the footing.

(c) When P and S are large, it is probable that a bigger stronger footing will be needed. Here the footing is depressed in order to get the benefit of cheap weight in the form of earth on top of TU. The anchor bolts are shorter, rods c tie the top together, and bars d are made strong enough to transfer the pull from the anchor bolts to the bottom of the footing and to withstand the bending at the base of the pedestal QR. Rods e are to resist any tension caused by uplift of one side of the footing. This tension may be caused by the weight of the footing itself and the earth on it. Although it may be small, this tension should be provided for when it is likely to exist. Rods f are for the normal action of the footing, to resist the overturning combined with the vertical load.

Example. Assume a footing like that of Fig. 8-11(c) used as part of the central bent of a small two-span steel overpass. Assume the following data: The net column load caused by the superstructure on the foundation is 160 kips minimum on the windward side with dead load and uplift only or 320 kips maximum when on the leeward side with dead, live, and wind

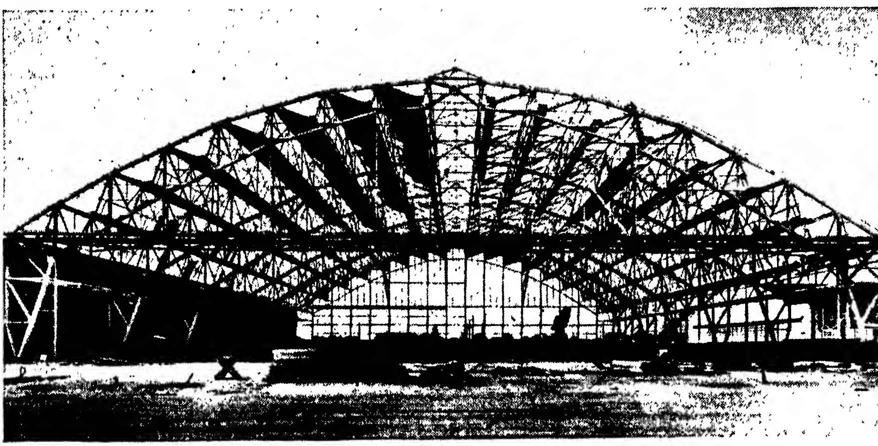


FIG. 8-11A. Final assembly hangar, Douglas Aircraft, Inc., Santa Monica, Calif. These are 3-hinged, trussed, rigid frames that cause an outward as well as a vertical force at their bearings. *Architects and Designing Engineers,* Edward Cray Taylor and Ellis Wing Taylor, Los Angeles, Calif. (*Courtesy of the Bethlehem Steel Co., builders of the steelwork.*)

loads combined; the steel base plate is 16 by 24 in.; S = 20 kips; the soil is 1 ft. 6 in. of sandy loam underlain by deep fine sand. Design the foundation for this column if the structure is near Rochester, N.Y.

From Table 4-5, assume that the allowable maximum pressure on the sand is 6 k.s.f. for $DL + LL$ and 7 k.s.f. for $DL + LL + W$. Because of frost, the footing will be placed so that YW is at least 5 ft. From the size of the base plate, the pedestal will be assumed to be 2 ft. 6 in. square. The anchor bolts are to hold the steelwork in position but not resist large overturning moments. Because of snow, the pedestal will be placed with its top 2 ft. above the ground.

How can one obtain a reasonable estimate of the size required for the footing? The following is one way to attack the problem:

1. The area at VW will exceed $320/7 = 46$ ft.^{2}, because of the weight of the foundation and the effect of overturning.

2. Use a slightly elongated footing to get greater resistance in the direction of S, and then add 8 or 10 ft.^{2} to this minimum area. Therefore, try $VW = 8$ ft., the width $b = 7$ ft., $WU = 2$ ft., and $UY = 3$ ft. Then the effective weight of the footing is computed as follows:

Pedestal $= 2.5 \times 2.5 \times 5 \times 0.150$	4.7
Footing $= 8 \times 7 \times 2 \times 0.150$	16.8
Earth $= (8 \times 7 - 2.5 \times 2.5)3 \times 0.1$	14.9
Total W'	36.4 kips

3.
$$M = 20 \times 7 = 140 \text{ ft.-kips}$$
$$e = \frac{M}{P + W'} = \frac{140}{320 + 36} = 0.39 \text{ ft.}$$

From Eq. (8-4),

$$p_1 = \frac{P + W'}{bL}\left(1 + \frac{6e}{L}\right) = \frac{356}{7 \times 8}\left(1 + \frac{6 \times 0.39}{8}\right) = 8.2 \text{ k.s.f.}$$

This is somewhat too large, since only 7 k.s.f. is allowed.

4. Try $L = 9$ ft. and $b = 7$ ft. Then $W' = 41$ kips and $e = 0.39$ ft.

$$p_1 = \frac{361}{9 \times 7}\left(1 + \frac{6 \times 0.39}{9}\right) = 7.2 \text{ k.s.f. (near enough)}$$
$$p_2 = \frac{361}{9 \times 7}\left(1 - \frac{6 \times 0.39}{9}\right) = 4.2 \text{ k.s.f.}$$

It is also essential to investigate this trial footing for the condition of minimum vertical load combined with the lateral force to see that it will not overturn. The magnitude of $P + W' = 160 + 41 = 201$ kips. Therefore,

$$e = \frac{M}{P + W'} = \frac{140}{201} = 0.7 \text{ ft.}$$
$$p_1 = \frac{201}{9 \times 7}\left(1 + \frac{6 \times 0.7}{9}\right) = 4.7 \text{ k.s.f.}$$
$$p_2 = \frac{201}{9 \times 7}\left(1 - \frac{6 \times 0.7}{9}\right) = 1.7 \text{ k.s.f.}$$

Since this case is found to cause less serious results than the maximum loading, proportion the footing to withstand the latter.

From the magnitudes and character of p_1 and p_2, it is apparent that no tensile reinforcement is needed in the top of the footing. The rods at the bottom may be determined and the footing analyzed upon the basis of the pressure diagram shown in Fig. 8-12(a). The upward shear at R for the entire rectangular area of the high-pressure side of the footing may be found from Sketch (b). The weight of the concrete footing and the soil on it above VW is 600 p.s.f. This may be deducted from the pressure diagram. Therefore,

$$V = \left[5.5 \times 3.25 + \left(\frac{1.1 \times 3.25}{2}\right)\right]7 = 138 \text{ kips}$$

Of course, the use of this shear in testing the punching shear along the edge of the pedestal at R will yield somewhat larger than the probable results, or the use of the trapezoidal area of bearing. However, using it as a trial gives

$$v_T = \frac{V}{bkd} = \frac{138{,}000}{30 \times 0.3 \times 21} = 730 \text{ p.s.i.}$$

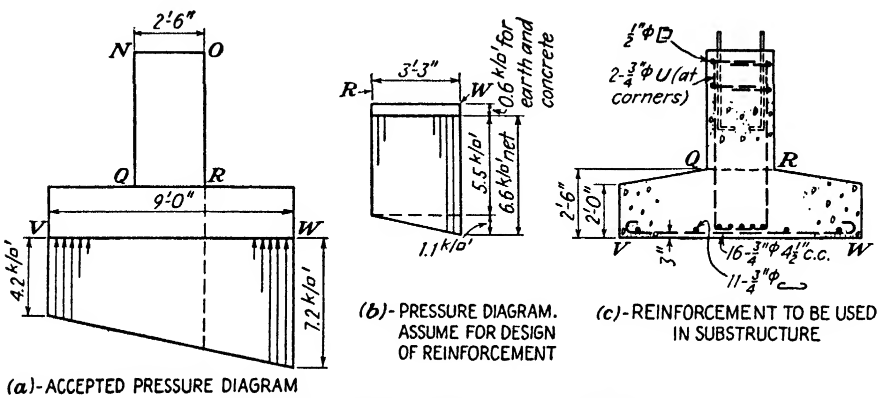


FIG. 8-12. **Pressure diagrams and reinforcement.**

This is so far above $0.2f'_c$ (600 p.s.i.) that the strength of the footing in shear should be increased. One way to do this is to slope the top of the footing so as to deepen it 6 or 8 in. at Q and R; another method is to slope the sides NQ and OR to reduce the cantilever; a third is to slope the other two sides of the pedestal so as to increase the resisting width; a fourth is to make the pedestal 3 ft. square or 3 ft. 6 in. square. The first method will be used because this will also increase the bending strength of the footing. The total depth will be increased to 2 ft. 6 in. Therefore, the new punching shear probably will not exceed

$$v_T = \frac{138{,}000}{30 \times 0.3 \times 27} = 570 \text{ p.s.i.}$$

This will be accepted.

The bending moment at R is

$$M = \left[5.5 \times 3.25 \times 1.62 + \left(\frac{1.1 \times 3.25}{2}\right)\frac{2}{3} \times 3.25\right]7 = 230 \text{ ft.-kips}$$

$$A_s = \frac{M}{f_s jd} = \frac{230}{18 \times 0.9 \times 2.25} = 6.3 \text{ in.}^2$$

$$\Sigma o = \frac{V}{ujd} = \frac{138{,}000}{150 \times 0.9 \times 27} = 38 \text{ in.}^2$$

Use sixteen ¾-in. round rods.

$$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 230{,}000 \times 12}{0.3 \times 0.9 \times 84 \times 27^2} = 330 \text{ p.s.i. (very safe)}$$

$$v_L = \frac{6.5 \times 1 \times 6{,}600}{84 \times 0.9 \times 23} = 25 \text{ p.s.i. (very safe)}$$

An arrangement for the reinforcement is shown in Fig. 8-12(*c*).

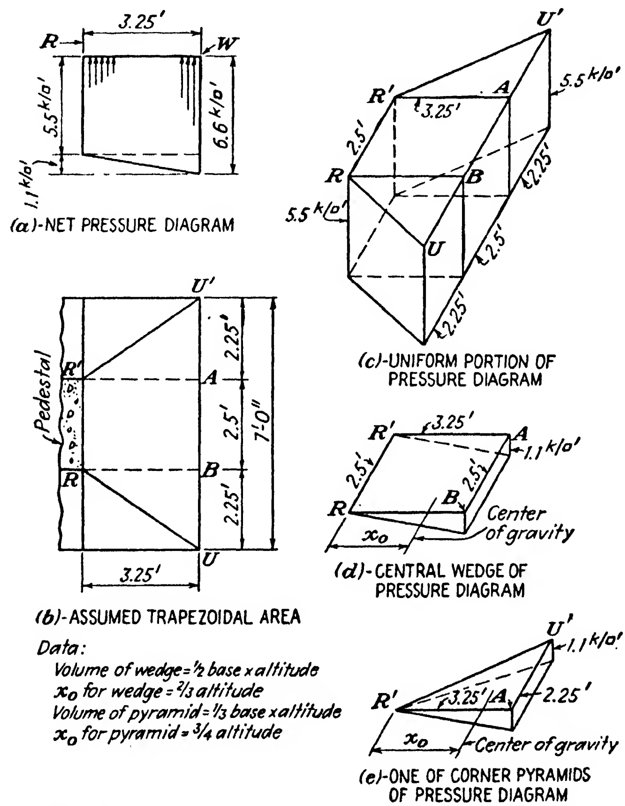


FIG. 8-13. Pressure diagrams for assumption of trapezoidal pressure area for analysis of footing.

If one wishes to compute the shear and bending at R of Fig. 8-12 upon the assumption of the trapezoidal area $RR'U'U$ of Fig. 8-13 (*b*) and the net pressure ordinate diagram shown in (*a*), this may be done by computing the volume of the assumed pressure diagram to find the shear, and the moment of this diagram about RR' to obtain the bending moment. The pressure diagram may be "cut" into the imaginary solid slice shown in Sketch (*c*), the wedge in (*d*), and two pyramids as in (*e*). The calculations are

Part	Volume		Lever arm		Moment
(c)	$2.5 \times 3.25 \times 5.5$	=	44.7×1.62	=	72.5
	$2(2.25 \times 3.25/2)5.5$	=	40.2×2.16	=	86.9
(d)	$2.5 \times 1.1 \times 3.25/2$	=	4.5×2.16	=	9.7
(e)	$2(2.25 \times 1.1/3)3.25$	=	5.4×2.43	=	13.1
	V	=	94.8 kips M	=	182.2 ft.-kips

These results are considerably less than those computed upon the rectangular basis. However, it is probable that this elongated footing sloping down at each end will act primarily as a beam cantilevered in one direction. The more tedious calculations, therefore, yield results of no special importance in this case.

The preceding methods of analysis are equally applicable to footings that are loaded eccentrically with respect to the rectangular bearing area. It makes no difference to the footing whether the direct load times an eccentricity causes the bending or whether the same M is due to other causes except that no increase in the allowable bearing pressure is advisable when the moment is caused by live and dead loads that are off center.

When overturning forces act simultaneously about two rectangular axes, the footing will tend to bear heavily on one corner. If there is compression under the entire footing, the maximum pressure may be approximated as follows:

$$p_1 = \frac{P}{A} + \frac{M_1c_1}{I_1} + \frac{M_2c_2}{I_2} \tag{8-8}$$

The minimum is

$$p_2 = \frac{P}{A} - \frac{M_1c_1}{I_1} - \frac{M_2c_2}{I_2} \tag{8-9}$$

If p_2 is theoretically tension, then p_1 may be approximated by trial about a diagonal axis, assuming triangular distribution and no tension.

In the discussion of Fig. 8-11, the resistance to sliding is assumed to be provided by friction on the bottom of the footing. One may properly ask "Will the resistance of the earth against a footing prevent its overturning?" In general, the abutting power of the soil—resistance to being pushed sidewise—may prevent actual overturning and sliding. However, the deformation of the soil and the rotation required to bring the earth into maximum resistance may be such as to cause uncertainty regarding how much of the stability is dependent upon unequal pressures under the footing and how much upon lateral resistance of the soil.

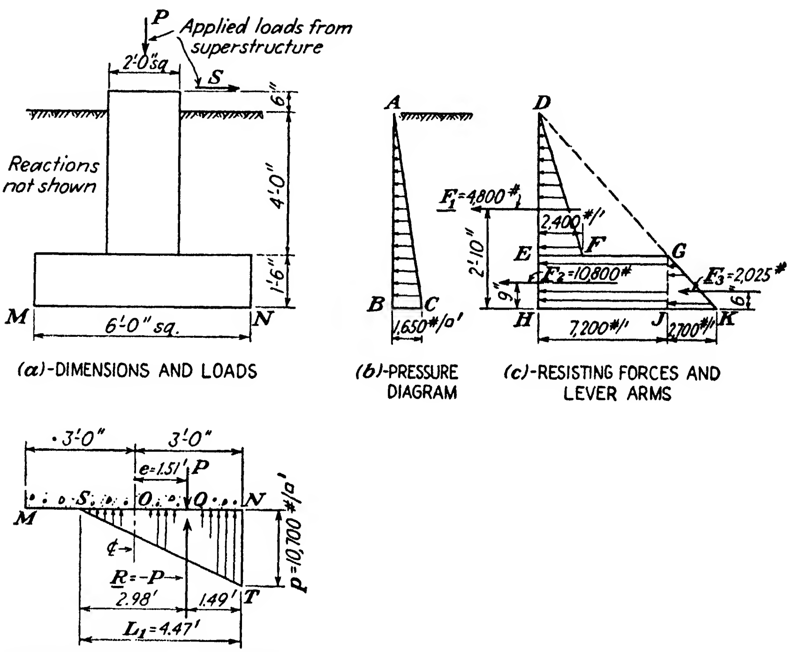
As an example of the theoretical lateral resistance that may be provided by the soil, assume the foundation shown in Fig. 8-14(a), and assume the gross load P = 144 kips and S = 40 kips. It is sufficient to use Rankine's formula for the lateral resistance of the soil, *i.e.*,

$$p = wh\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right) \tag{8-10}$$

where p = resistance in pounds per square foot, w = unit weight of earth (assumed here as 100 p.c.f.), h = depth in feet to the point considered, and ϕ = angle of internal friction of the soil, assumed to be 30°. Therefore,

$$p = 100h\left(\frac{1 + 0.5}{1 - 0.5}\right) = 300h \tag{8-11}$$

Thus, the assumed triangular pressure diagram for a vertical strip of soil 1 ft. wide will be as shown in Sketch (*b*). The computed diagram of total



(*a*)-DIMENSIONS AND LOADS (*b*)-PRESSURE DIAGRAM (*c*)-RESISTING FORCES AND LEVER ARMS

(*d*)-PRESSURE DIAGRAM UNDER FOOTING

FIG. 8-14. Approximation of maximum lateral supporting power of earth on an isolated footing subjected to overturning.

pressure on the 2-ft. square pedestal is then *DEF* of Sketch (*c*), whereas that on the 6-ft. wide footing is *EGKH*. Summing up the lateral resistances, the soil will withstand a lateral force of approximately 17.6 kips, leaving 40 − 17.6 = 22.4 kips to be resisted by friction under *MN*. The required coefficient of friction is, therefore,

$$22.4 = 144f \qquad \text{or} \qquad f = 0.16 \text{ (very safe)}$$

The moment of these lateral resistances about point *N* is

DEF, 4.8 × 2.83	13.6
EGJH, 10.8 × 0.75	8.1
GJK, 2.0 × 0.5	1.0
Total	22.7 ft.-kips

The resultant overturning moment that must be resisted by the footing MN is then

$$40 \times 6 - 22.7 = 217.3 \text{ ft.-kips}$$

The corresponding eccentricity of the resultant at MN is

$$\frac{M}{P} = \frac{217.3}{144} = 1.51 \text{ ft.}$$

This is shown in Fig. 8-14(d), and $L_1 = 4.47$ ft. From Eq. (8-6),

$$p_1 = \frac{2P}{3b[(L/2) - e]} = \frac{2 \times 144}{3 \times 6(3 - 1.51)} = 10.7 \text{ k.s.f.}$$

From the preceding computations, it is apparent that the benefit of the surrounding earth in resistance to sliding is considerable, whereas it is relatively small as far as overturning is concerned unless the substructure is wide and deeply embedded. It is generally advisable to design foundations subjected to overturning so that they will be safe without dependence upon the lateral resistance of the soil. On the other hand, exceptions occur. For example, piles in mud, deep bridge piers in clay, narrow cylindrical caissons in silt, and tall pedestals extending up through earth fill are cases in which the lateral supporting power of the soils may be relied upon safely.

One may compute the end restraint at the base of a concrete or steel member, and he may depend upon the footing to develop the required resisting moment. Theoretical calculations may show that the selected footing can offer this resistance safely. However, when the footing is on weak or plastic soil and when the overturning moment is applied steadily or for a relatively long time, compression and flow of the soil caused by the large edge pressure under the footing may produce sufficient angular rotation to eliminate the assumed restraint. The angular movement required to transform the fixed end of a stiff member into what approximates a hinged end is often small. This should be borne in mind when rigid-frame structures are designed.

What does the embedment of a foundation have to be in order to enable one to rely upon the lateral support (passive resistance) of the surrounding earth to prevent overturning? This is almost impossible to answer. There is no sharp dividing line between the depth that is sufficient and the one that is not. The following are given as a general aid in drawing one's own conclusions:

1. An appreciable deformation of ordinary earth, especially backfill around a foundation, will occur before a large passive resistance is developed. Coarse sand and gravel are fairly dependable, but clay, silt, and fine sand may yield too much.

2. If a slight angular rotation of the substructure is permissible, the resistance of the surrounding soil will probably come into action strongly

before failure by tipping can occur. However, appreciable movement is not generally permissible.

3. When failure would be serious, one should be careful about depending upon such uncertain resistances.

4. Consider the possibility of someone in the future unwittingly removing the soil upon which the structure depends for lateral support.

5. The depth for reliability is relative. For example, an embedment of 10 ft. is a lot for an isolated pedestal and footing 5 ft. square; it is very small for a bridge pier 60 ft. high with a base 15 by 40 ft. in area.

There may be cases in which a substructure that must resist overturning is supported directly upon bare rock. The rock will probably withstand safely more pressure than will the concrete, hence the magnitude of the pressure is unimportant. Sliding may be resisted by roughening the rock surface if it is not already so. Overturning, however, must be withstood by the substructure itself.

As an illustration, consider the stepped footing in Fig. 8-15(a), which is to support a steel post used in the power lines for the electrification of a railroad. Assume that there is a reversible overturning moment $M = 100$ ft.-kips, a horizontal shear $S = 6$ kips, a gross load $P = 25$ kips, and that $L/2 = 4$ ft. and $h = 5$ ft. Taking moments about A, the overturning moment is

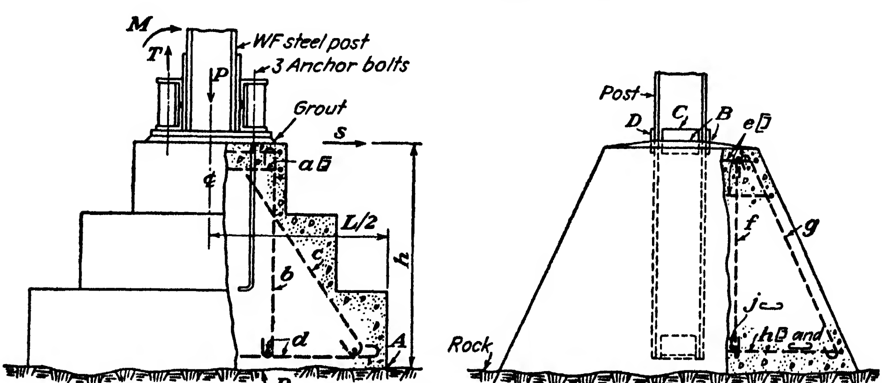
$$M_O = 100 + 6 \times 5 = 130 \text{ ft.-kips}$$

whereas the righting moment is

$$M_R = 25 \times 4 = 100 \text{ ft.-kips}$$

Therefore, the footing is not safe. If the footing is increased to 11 ft. in width and P to 45 kips,

$$M_R = 45 \times 5.5 = 250 \text{ ft.-kips}$$



(a)-A STEPPED FOOTING (b)-A TRAPEZOIDAL FOOTING

FIG. 8-15. Two types of foundation on rock for railroad electrification.

The safety factor against overturning is then 250/130 = 1.9, which is satisfactory.

It is apparent from the above that these footings must be very broad and heavy. The pull on the anchor bolts will also be large. If the post is embedded in the footing, as pictured in Fig. 8-15(*b*), the lateral pressure at *B* may be important. Here the concrete is sloped up to help reduce corrosion of the steel. Rust plates *C* and *D* may help to protect the main material. The reinforcement shown in Fig. 8-15 is designed to knit the concrete together and permit the footings to "ride either edge" safely. If *M* and *S* are not reversible, an unsymmetrical footing may be useful, but it may not result in important economies because most of *P* is caused by the weight of the footing itself.

One might well ask why the steelwork of Fig. 8-15(*a*) should not be anchored to or in the rock itself. This possibility and the difficulties encountered are discussed in connection with backstays in Art. 8-10.

FIG. 8-16. A steel transmission tower that was damaged when the boom of a railroad crane collided with it in the darkness. Notice the crack in the foundation.

Figure 8-16 shows construction that was made for the electrification of the railroad at a mine. The steelwork was erected during a certain day. That night the operator of a railroad crane was ordered to take his machine out to a particular place, but he was not told about the new overhead trusses. In the dark, his boom collided with the steelwork and caused the wreckage shown. Notice how the heavy base details held together, and how the tensile shock delivered to the footing by the anchor bolts caused the concrete to crack in two. The footing is shown in Fig. 8-17, and the approximate location of the crack is indicated. Of course, footings are not designed—and cannot be economically—for such conditions. However, as a general principle, it is desirable to make them sufficiently strong to cause the superstructure to fail without wrecking the foundation. If only the former fails, it may be patched up; if the latter gives way, then both parts have to be rebuilt.

8-7. Unsymmetrical footings. Occasionally a foundation is to be unsymmetrical, and the center of gravity of the bearing area will not coincide with the resultant of the vertical loads. This may be caused by the limitations of space and by interference of pipes, pits, and machinery foundations with footings. Eccentricities of varying magnitudes and direc-

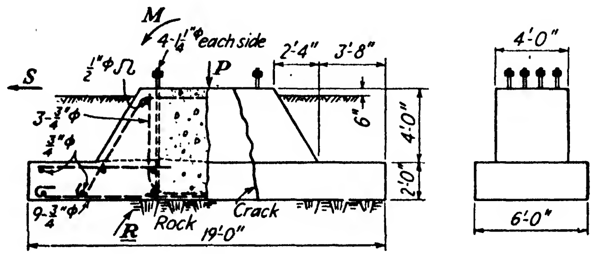


FIG. 8-17. Footing that cracked.

tions may be caused by large movable loads that may be in one location at one time and in a different place at another; for example, an interior footing in a factory may support separate columns under two adjoining crane runways, as pictured in Fig. 8-52. It sometimes happens, too, that alterations make it desirable to cut a piece off from an existing footing. It is desirable, therefore, to be sure that the pressures are not excessive and that these foundations are safe. Heavy loads times moderate eccentricities may produce large moments and surprisingly big edge pressures. Figure 8-17*A* pictures a case in which a one-foot strip has been cut off along side *AB*. The load *P* will now be eccentric.

The eccentricity of a load causes the equivalent of an overturning moment. Therefore, the method of analysis of an unsymmetrical foundation is similar to the procedures described in the preceding article.

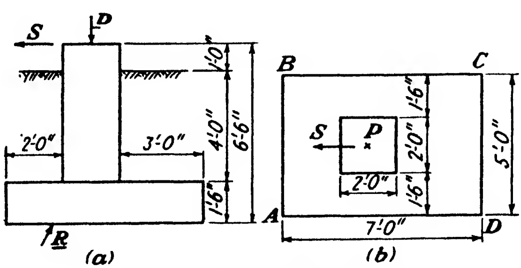
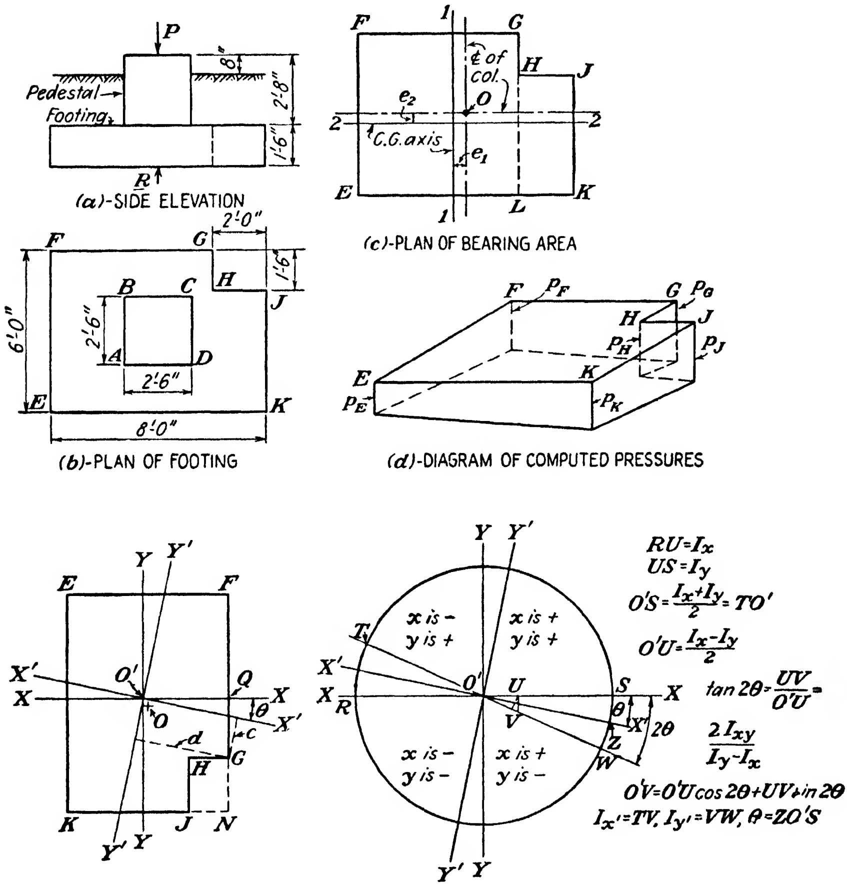


FIG. 8-17A. Footing with eccentric load.

As an illustration of the method of analysis, assume the footing shown in Fig. 8-18. It is for an interior column in an industrial plant. One corner is to be cut as shown in Sketch (*c*). Assume that $P = 275$ kips. Compute the gross pressures under the footing, and draw the pressure diagram.

The gross weight W' of the footing and earth is computed as follows:

Footing = (48 − 3)225	10,100
Pedestal = 6.25 × 2.67 × 150	2,500
Earth = (48 − 3 − 6.25)200	7,800
Total W'	20,400 lb. (say 21 kips)



(e)-LOCATION OF PRINCIPAL AXES

(f)-GRAPHICAL METHOD FOR LOCATING PRINCIPAL AXES AND DETERMINING PRINCIPAL MOMENTS OF INERTIA

FIG. 8-18. Analysis of pressure under an unsymmetrical footing.

The eccentricities are

$$e_1 = \frac{-2 \times 1.5 \times 3}{48 - 3} = -0.2 \text{ ft., or } 0.2 \text{ ft. from } O \text{ for axis 1-1}$$

$$e_2 = \frac{-3 \times 2.25}{45} = -0.15 \text{ ft., or } 0.15 \text{ ft. from } O \text{ for axis 2-2}$$

Therefore, about the center of gravity axes,

$$I_1 = \frac{6 \times 8^3}{12} - \frac{1.5 \times 2^3}{12} - (3 \times 3^2) - (45 \times 0.2^2) = 226 \text{ ft.}^4$$

$$I_2 = \frac{8 \times 6^3}{12} - \frac{2 \times 1.5^3}{12} - (3 \times 2.25^2) - (45 \times 0.15^2) = 127 \text{ ft.}^4$$

Assuming P and W' to be practically colinear, the overturning moments are

$$M_1 = (P + W')e_1 = 296 \times 0.2 = 59 \text{ ft.-kips}$$
$$M_2 = (P + W')e_2 = 296 \times 0.15 = 44 \text{ ft.-kips}$$

From the use of Eq. (8-4) in its general form, the computed corner pressures are

$$p_E = \frac{296}{45} - \frac{59 \times 3.8}{226} - \frac{44 \times 2.85}{127} = 4.6 \text{ k.s.f.}$$

$$p_F = \frac{296}{45} - \frac{59 \times 3.8}{226} + \frac{44 \times 3.15}{127} = 6.7 \text{ k.s.f.}$$

$$p_G = \frac{296}{45} + \frac{59 \times 2.2}{226} + \frac{44 \times 3.15}{127} = 8.2 \text{ k.s.f.}$$

$$p_H = \frac{296}{45} + \frac{59 \times 2.2}{226} + \frac{44 \times 1.65}{127} = 7.7 \text{ k.s.f.}$$

$$p_J = \frac{296}{45} + \frac{59 \times 4.2}{226} + \frac{44 \times 1.65}{127} = 8.2 \text{ k.s.f.}$$

$$p_K = \frac{296}{45} + \frac{59 \times 4.2}{226} - \frac{44 \times 2.85}{127} = 6.7 \text{ k.s.f.}$$

The pressure diagram is pictured in Fig. 8-18(*d*).

The reinforcement of the footing in Fig. 8-18(*b*) may be made in the form of one strong band perpendicular to AD and BC, and another running across AB and CD. For computing the bending moments and shears, the methods explained in connection with Fig. 8-13 may be used when the variations of pressure are too great to justify the use of a uniform averaged intensity on any given portion of the footing.

By similar procedures, the pressure may be approximated for various combinations of forces and conditions that produce unequal bearing as long as none of the pressures are theoretically negative. If the assumed or actual loads and conditions cause serious variations of bearing with a tendency for large uplift under a corner and an appreciable area of the footing, an engineer might well spend time and thought first upon the best way to change the basic construction so as to eliminate such questionable construction.

The preceding computations may not yield the critical magnitudes of the pressures under the footing. To obtain these in cases where it seems to be desirable to do so, the principal axes and moments of inertia should be used. The procedure then is the following, referring to Fig. 8-18:

1. Draw the footing in outline as in Sketch (*c*) or (*e*). The latter will be used purposely to illustrate this.

2. Compute the bearing area of the footing, locate its center of gravity, and draw the rectangular axes X-X and Y-Y through the center of gravity O'.

3. Assume that positive angles are to be measured counterclockwise from $O'Q$ to locate the principal axes of inertia.

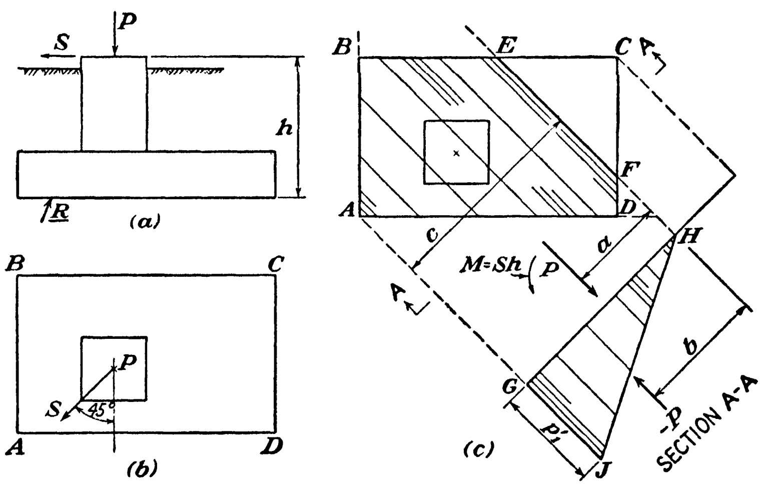


FIG. 8-19. Footing with unsymmetrical loads and overturning.

4. Compute the moments of inertia I_x and I_y about the chosen rectangular axes.

5. Compute the product of inertia I_{xy} about the center of gravity, bearing in mind the signs of the coordinates.

6. From the equation

$$\tan 2\theta = \frac{2I_{xy}}{I_y - I_x} \tag{8-11a}$$

obtain the angle θ to locate the principal axes X'-X' and Y'-Y'. If $\tan 2\theta$ is positive, measure θ above axis X-X; if negative, measure it clockwise from $O'Q$. Of course, if the tangent of 2θ is negative, the angle can be measured counterclockwise from $O'Q$ into the second quadrant. Then θ will locate one of the principal axes as before, neglecting any uncertainty as to which is X'-X' and which is $Y'Y'$. Measuring clockwise from $O'Q$ for negative values of $\tan 2\theta$ may avoid some confusion. However, the other method is used purposely in Fig. 14-39.

7. The principal moments of inertia are

$$I_{\max} = \frac{I_x + I_y}{2} + \frac{I_x - I_y}{2} (\cos 2\theta) - I_{xy} \sin 2\theta \tag{8-11b}$$

$$I_{\min} = \frac{I_x + I_y}{2} - \frac{I_x - I_y}{2} (\cos 2\theta) + I_{xy} \sin 2\theta \tag{8-11c}$$

8. Compute the components of the overturning moments normal to the directions of the principal axes. Call these $M_{x'}$ and $M_{y'}$, respectively.

9. Compute or scale the lever arms from the principal axes to points at which the pressure is desired, as c and d in Sketch (e).

10. Compute the intensity of pressure under seemingly critical points, such as G of (e), as follows:

$$p_G = \frac{P}{A} \pm \frac{M_{x'}c}{I_{x'}} \pm \frac{M_{y'}d}{I_{y'}} \tag{8-11d}$$

With the values previously computed for the footing of Figs. 8-18(c) and (e), the computations for critical pressures at G and J are the following:

$$I_{xy} = 48(-0.2)(+0.15) - 2 \times 1.5(-3.2)(+2.4) = +21.6 \text{ ft.}^4$$

$$\tan 2\theta = \frac{2(21.6)}{127 - 226} = -0.437$$

$$2\theta = -(23°37')$$

$$\sin 2\theta = -0.401 \qquad \cos 2\theta = +0.916$$

$$\theta = -(11°48') \qquad \sin\theta = -0.204 \qquad \cos\theta = +0.979$$

$$I_{\max} = \frac{226 + 127}{2} + \frac{226 - 127}{2}(0.916) - 21.6(-0.401) = 230 \text{ ft.}^4$$

$$I_{\min} = \frac{226 + 127}{2} - \frac{226 - 127}{2}(0.916) + 21.6(-0.401) = 123 \text{ ft.}^4$$

$$M_x = 59 \text{ ft.-kips} \qquad \text{and} \qquad M_y = 44 \text{ ft.-kips}$$

Remember that the load acts at O, Sketch (e). Then, neglecting signs of angles but considering directions of rotation, the new coordinates of point O are

$$x' = 0.2 \sin\theta + 0.15 \cos\theta = 0.188$$

$$y' = 0.2 \cos\theta - 0.15 \sin\theta = 0.165$$

$$M_{x'} = 296 \times 0.165 = 49 \text{ ft.-kips}$$

$$M_{y'} = 296 \times 0.188 = 56 \text{ ft.-kips}$$

Scaling the lever arms from Fig. 8-18(e), or computing them if desired, the estimated pressures at G and J are

$$p_G = \frac{296}{45} + \frac{49 \times 1.5}{230} + \frac{56 \times 3.6}{123} = 8.5 \text{ k.s.f.}$$

$$p_J = \frac{296}{45} + \frac{49 \times 3.75}{230} + \frac{56 \times 2.5}{123} = 8.5 \text{ k.s.f.}$$

These values are only slightly greater than the 8.2 k.s.f. computed previously. However, this may not always be the case.

The Mohr-Land circle[1] is a graphical method for obtaining the positions of the principal axes and for finding the principal moments of inertia. Referring to Fig. 8-18(*f*), the procedure is the following:

1. Draw a circle with radius $\frac{I_x + I_y}{2}$, using the values of I_x and I_y about any rectangular axes such as X-X and Y-Y of Fig. 8-18(*e*).
2. Scale off RU to equal I_x, and US to equal I_y.
3. Compute I_{xy} as for (*c*).
4. From U, draw $UV = I_{xy}$, and normal to $O'S$. If $\tan 2\theta$ is positive, draw UV above $O'S$; if negative, draw it below $O'S$ as for this problem.
5. $$O'U = \frac{I_x - I_y}{2}$$

Therefore,

$$\frac{UV}{O'U} = I_{xy} \div \frac{I_x - I_y}{2} = \frac{2I_{xy}}{I_x - I_y} = \tan 2\theta$$

Then $2\theta = SO'W$ and $\theta = SO'Z$.

6. $$TV = I_{max}$$ $$VW = I_{min}$$

The preceding methods of analysis assume that there is compression under the entire area of the footing. If this is not the case, they are inapplicable.

When necessary, one may compute the pressures under a footing subjected to eccentricities and a serious diagonal overturning, as indicated in Figs. 8-19(*a*) and (*b*) by trial. A rectangular footing is shown for simplicity. The procedure is outlined as follows:

1. By means of the general application of Eq. (8-4), compute the pressure diagram as though there were tension under part of the footing.
2. By graphics or algebra locate the position of the line of zero pressure under the footing.
3. Select a trial zero axis, parallel to the one in item 2 but somewhat nearer the high-pressure corner, as shown by EF in Sketch (*c*).
4. Draw a plan of the footing to scale, and show area $ABEFD$.
5. Using the plan view as in Sketch (*c*), scale off the perpendicular distances from A, B, and D to the axis EF. Then compute the moment of inertia of area $ABEFD$ about axis EF, using $I = bh^3/12$ for a large triangle with its vertex at A and base along EF minus a similar I for each of the triangles with vertices at B and D.

[1] Courtesy of Prof. C. J. Posey and Mr. Angel Lazaro, Jr., State University of Iowa, Iowa City, Iowa.

6. Assume the total overturning moment $M_t = Pa + Sh$, equate this to $p_1 I_{EF}/c$, and solve for p_1.

7. Assume that HJ of section A-A in Sketch (c) is a straight line. From it the pressure diagram may then be determined sufficiently for practical purposes. Check to see if the volume of this pressure diagram approximately equals the total load P. If the discrepancy is large, assume a new location for the axis EF, and try again.

8. If the bearing area is very irregular, I_{EF} may be approximated by dividing it into arbitrary strips as indicated in (c). From $I + Ax^2$ for each strip modified into a more or less equivalent rectangle or triangle, a value for I_{EF} may be obtained.

8-8. Footings steadied laterally by structural parts. Except for the lateral resistance provided by surrounding soil, the preceding discussions have assumed that a footing must resist rotation by unequal pressures under its base. What happens when the substructure, or the pedestal, is incorporated in the structure so that its top is steadied? It may be that a resistance is provided that will prevent overturning so that the footing should be designed for vertical loads only.

Assume the exterior column footing shown in Fig. 8-20. It is subjected to load P together with a reversible overturning moment M and a shear S.

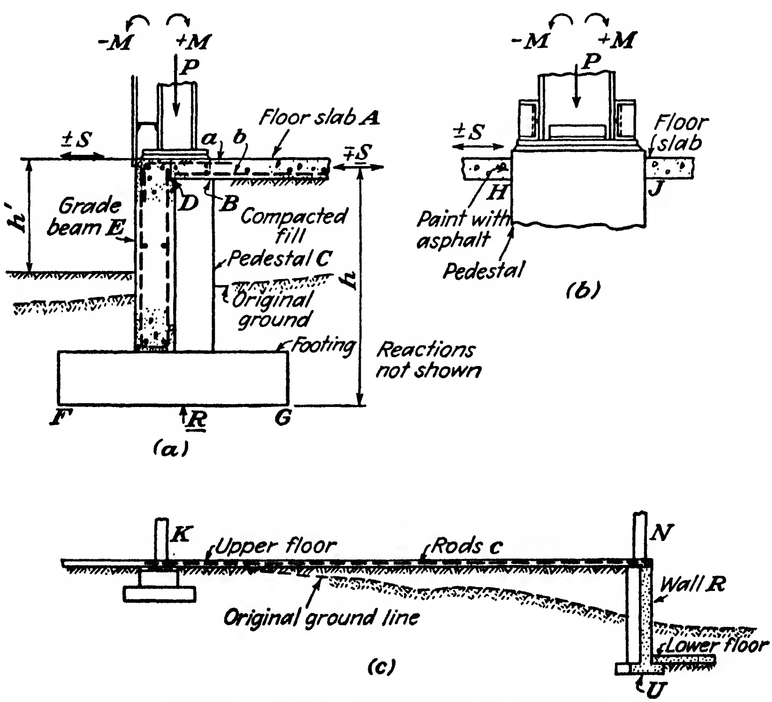


FIG. 8-20. Walls and footings steadied by floors.

The concrete floor slab is seated upon the pedestal C and the grade beam E at B and D, respectively. Rods a tie the floor to the foundation. Of course, the floor will not contribute appreciably to the support of load P and the substructure; the reverse is probable. The shear S, however, will be transmitted by direct pressure or by tension in rods a to the floor slab. If the frictional resistance of this floor on the ground is sufficient or if the floor is connected to anything that will serve as an adequate anchorage, the shear S will not cause the footing to overturn. Furthermore, if the bottom of the foundation is embedded and if the floor can hold the top of the pedestal steady, the moment M can cause no angular movement of the substructure, hence it will be unlikely to do more than add to the force S another lateral force $S' = M/h$. Without rods a, tensile resistance is unreliable; without the shelf, vertical loads on the floor slab may bend this reinforcement or spall the concrete because of possible settlement of the floor.

How much of a floor can be relied upon to resist lateral forces? Assume that the columns are 22 ft. c.c., the nearest real contraction joint in the floor of Fig. 8-20(a) parallel to beam E is 20 ft. away, the floor is 6 in. thick, and h' equals 3 ft.

The soil for approximately $1.5h'$ back from E depends largely upon the grade beam and footing for its lateral support. Therefore, reliance upon this strip of soil to support E and C is unsafe. The weight of the effective concrete of the slab is, approximately,

$$22 \times 15 \times 75 = 25{,}000 \text{ lb.}$$

With a coefficient of friction of 0.7 and a safety factor of 1.5, the tension that can be relied upon in rods a is

$$T = \frac{25{,}000 \times 0.7}{1.5} = 11{,}600 \text{ lb.}$$

Of course, the grade beam should be keyed to the pedestal, and no transverse contraction joint should be located so that it will prevent the intended action.

In an industrial plant, the floor was not supported upon the pedestals of interior column foundations but was made as shown in Fig. 8-20(b). No thick compressible fillers were used at H and J. Therefore, the top of the foundation could not rotate without pressing against the floor at H or J. Since the floor was sufficiently heavy and large in area, it was relied upon to resist the effects of M and S.

In another plant, there was a long depressed pit or lower floor 10 ft. deep, as shown in simplified manner in Fig. 8-20(c). Lateral loads from columns N and from the earth backfill behind wall R were resisted by rods c that tied this wall through the floor to the foundations of columns K. The vertical loads from columns N were distributed to the soil by wall R and its footing U. The earth pressures on the bottom of R were resisted by both the earth and the lower floor bearing against U.

In many instances it is possible and desirable to plan a structure so that large overturning forces upon footings can be avoided by the action of other parts of the structure. In some cases, this is not safe because of the likelihood that the supporting floors or parts may be removed or partly incapacitated in the future. In still others, it is desirable to plan the foundations so that they are sufficient by themselves. Which course to pursue is a question that requires an answer based, at least partly, upon good engineering judgment.

8-9. Footings subjected to uplift and overturning. The footings of tall light structures may be subjected to such relatively large overturning moments that the foundations on one side must resist considerable uplift. In some instances, the uplift may not overcome the downward dead loads, but the resultant vertical force may be so small that the footings are not able effectively to resist horizontal shearing forces and tipping. These foundations may need special care in their design to assure safety. The principles involved will be illustrated by means of a few specific structures.

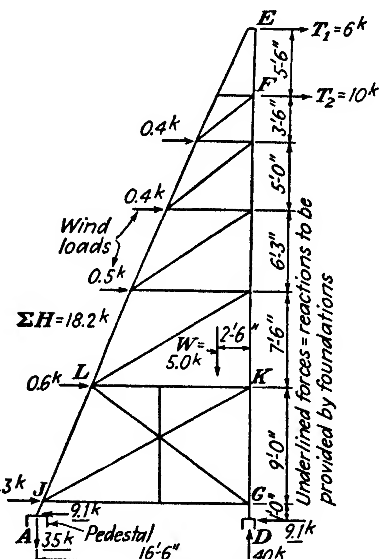


FIG. 8-21. Dimensions and loads for one bent of a dead-end tower.

Suppose that a high-tension power line is being built to supply electric power to a new factory. The cables are to be supported by a steel dead-end tower at the substation. Figure 8-21 shows the dimensions and assumed forces acting upon one bent of this tower. There are two such bents 12 ft. c.c., and the front and back of the tower are both braced strongly. The front bracing in the plane of GK is a portal frame so that a large transformer may be placed inside the tower. Two other transformers are to be installed, one on each side of the tower. How may the foundations be designed to resist the required forces? Their resistance to transverse wind loads should be tested after the substructure has been designed to withstand longitudinal wind and the cable pulls T_1 and T_2. The resultant of the dead load of the bent and the cables is W. Assume that the soil is sandy and able to support a bearing pressure of 7 k.s.f.

It is obvious that the tower will tend to tip about point D even though the trussing terminates at G in order to simplify the steel details. Taking moments of all forces about D, the reaction at A is

$$R_A \times 16.5 = 6 \times 37.75 + 10 \times 32.25 + 0.4 \times 28.75 + 0.4 \times 23.75 + 0.5 \times 17.5 + 0.6 \times 10 + 0.3 \times 1 - 5 \times 2.6$$

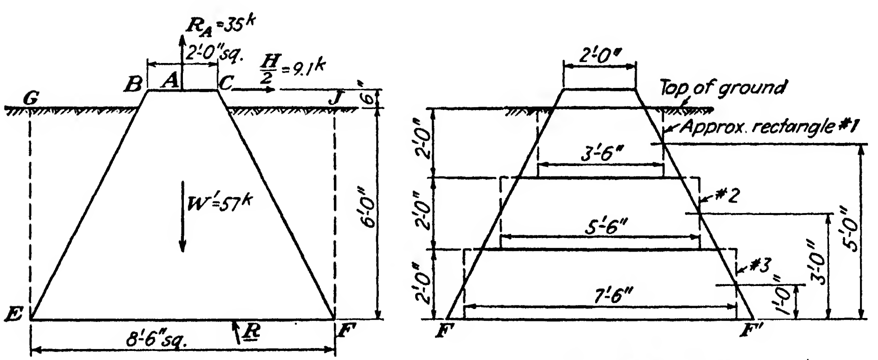
$$R_A = 35 \text{ kips uplift}$$

$$R_D = 40 \text{ kips downward}$$

If the cable pulls T_1 and T_2 are the ultimate strengths of the wires, the safety factor against overturning may be some such value as 1.25; if these pulls are the maximum probable forces caused by dead load, ice, and wind—as they are in this problem—the safety factor of the tower and its foundation should be such that the cables will almost certainly fail before the tower will collapse or tip over.

Assume that the safety factor is to be 1.6. Then the downward force required at A is $35 \times 1.6 = 56$ kips. Furthermore, if the tower members shown in Fig. 8-21 are properly designed, the shear H will be divided between A and D if the foundation of each can resist these forces. For design purposes, assume 9.1 kips at each pedestal. If member JG is not supplied in the tower itself, a concrete strut between A and D may serve a similar equalizing purpose. If members JG and LG are both omitted, the bearing at A must transmit the entire shear into the substructure.

Many different foundations may be used for this tower. Figure 8-22 shows one type of isolated footing that will be considered. The rear footing in Sketch (a) is a trapezoidal or pyramidal concrete block. The soil around



(a)-REAR FOOTING (b)-SIDE ELEVATION AND COMPUTATION OF RESISTANCE OF SOIL

Part		Force in lb.		Lever arm, ft.		Moment, ft-lb.
1	3.5 x 2 x 300 =	2,100	x	5	=	10,500
2	5.5 x 2 x 900 =	9,900	x	3	=	29,700
3	7.5 x 2 x 1,500 =	22,500	x	1	=	22,500
	Σ =	34,500			Σ =	62,700

FIG. 8-22. A trapezoidal footing for support of uplift at rear of a dead-end tower.

the sides and above the edges EF will be included as useful weight although the wedging action of the sloping sides may cause one to question seriously any reliance upon this ground above E and F. By trial, choose dimensions for the footing until the weight of the concrete and earth equals at least the desired 56 kips. The concrete footing shown in the figure weighs approximately 37 kips; the earth within $GEFJ$, 20 kips; the total, 57 kips. This footing will be accepted tentatively for further analysis.

The net downward load at A is $57 - 35 = 22$ kips. Then, applying the safety factor of 1.6, $M = 1.6 \times 9.1 \times 6.5 = 94$ ft.-kips, and $e = 94/22 = 4.3$ ft. Therefore, the footing cannot offer the necessary resistance without being about to turn over.

If the abutting power of the earth is included, with unit resisting pressures of $300h$ acting upon the side CF of the footing, the moment of these resistances about F is approximately 63 ft.-kips, and the resisting force is 34 kips, as indicated in Sketch (*b*). If the soil can be relied upon, it can resist much of the overturning moment and all of the shear. The moment to be resisted by the bearing at EF is $94 - 63 = 31$ ft.-kips, and $e = 31/22 = 1.4$ ft. Then,

$$p_1 = \frac{22}{8.5 \times 8.5}\left(1 + \frac{6 \times 1.4}{8.5}\right) = 0.6 \text{ kip (approx)}$$

$$p_2 = 0 \text{ (approx)}$$

It does not seem wise to depend upon the lateral resistance of the earth to such a large extent when failure of the structure might shut down the plant, and cause loss of life, too. Therefore, although the footing at D may be made satisfactory in itself, it would have to be large to resist overturning and, since it is desirable to investigate a different type of foundation at A, a safer design will be sought for both.

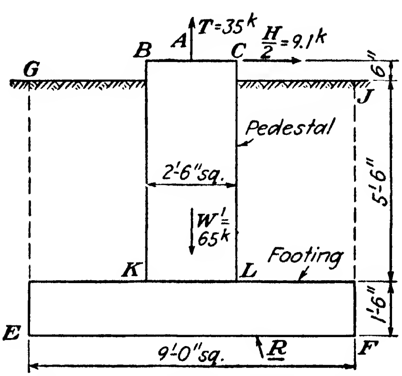


FIG. 8-23. A proposed isolated spread footing.

In Chap. 4, the shearing resistance of soil is assumed to be utilizable in spreading the pressure from a localized load to the lower strata. Why cannot this same resistance be depended upon in the case of uplift and overturning, such as for the footings in Figs. 8-22 and 8-23? This would mean that the lines EG and FJ in Fig. 8-22, for example, would slope upward and outward from the base EF at the angle of internal friction to bound a frustum of an imaginary pyramid with its small end at the bottom. The theoretical weight of the enclosed earth for resistance to uplift or overturning would be increased. Even with movement along the vertical planes

EG and *FJ*, one might argue that there will be shearing or frictional resistance to upward movement of the enclosed mass. In actuality, the soil may tend to behave this way, but it seems advisable to rely only upon the weight of the soil vertically above the base unless the embedment is perhaps 10 ft. or more, because the surrounding material is probably backfill. The economies secured otherwise may be small in comparison to the seriousness of failure.

Would the foundation shown in Fig. 8-23 be satisfactory? The footing is made somewhat deeper and wider than the bottom of Fig. 8-22(*a*), and the total weight is approximately 65 kips. However, a comparison with the analysis of the preceding trapezoidal footing shows that this one is not sufficiently broad and heavy to remedy the overturning danger, and the narrow pedestal will offer even less sideward bearing resistance against the soil. The ground that is actually in contact with the concrete pedestal is, of course, backfill, and it may be rather compressible. The soil on top of the footing but beside the pedestal cannot be relied upon for lateral resistance even though its weight is useful for stability. These isolated footings are more suitable when the chords of the tower trussing are spread much farther apart. Therefore, look for a design that is inherently more satisfactory.

In Fig. 8-24(*a*) the pedestals at *A* and *D* under each side of the dead-end tower of Fig. 8-21 are placed upon a common footing. The reactions at *A*

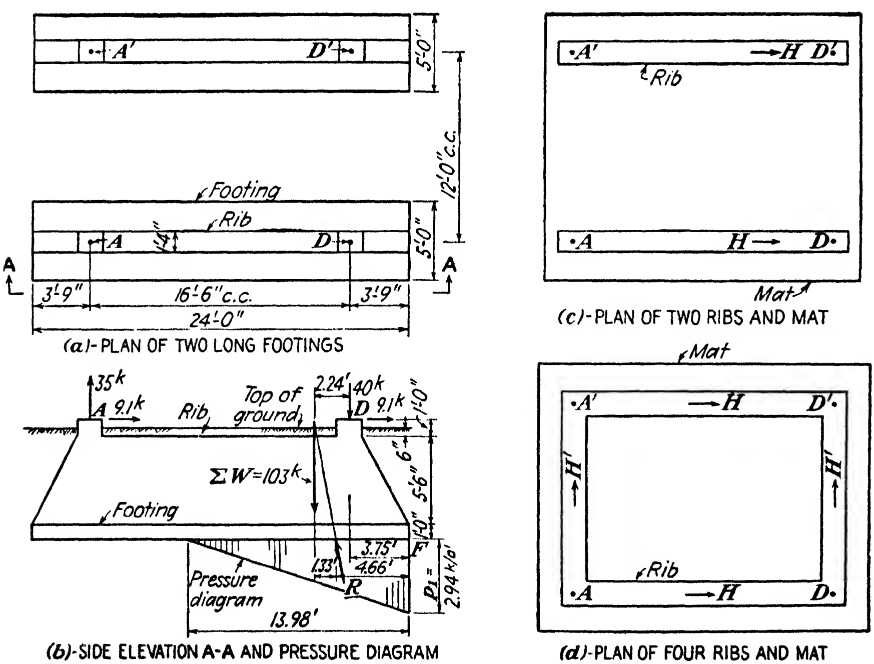


FIG. 8-24. Combined footings for a dead-end tower.

and D cause a tendency to break off the footing between them, and also (indirectly) the part in front of point D. When the forces are large, it is desirable to use a rib under the bearings as shown in Sketch (*b*), the footing merely being a cantilever to resist upward pressure in front and downward weight in the rear. A floor to support a transformer within the tower may be placed across the two ribs, but this will not be counted upon for stability. For the dimensions shown in (*b*), $W' = 98$ kips. The safety factor (S.F.) against overturning about F is

$$\text{S.F.} = \frac{98 \times 12 + 40 \times 3.75}{35 \times 20.25 + 18.2 \times 7.5} = 1.57$$

The resultant vertical load is $98 + 40 - 35 = 103$ kips, and is 2.24 ft. from D. The pressure diagram is shown in Sketch (*b*). Friction alone under the footing is able to resist sliding without dependence upon the abutting power of the earth in front of the substructure. This design seems to be a more satisfactory solution.

A possible modification of this design is that in Sketch (*c*) where the two footings are joined as a single mat. This may be desirable if the tower is sufficiently narrow. When large forces may be in two directions, as indicated in (*d*), and when H' may be reversible, the four-rib and mat construction shown may provide a simple and reliable foundation.

The construction shown in Fig. 8-25 is still another variety of foundation for this tower. It consists of a heavy cross wall and footing at the rear, a

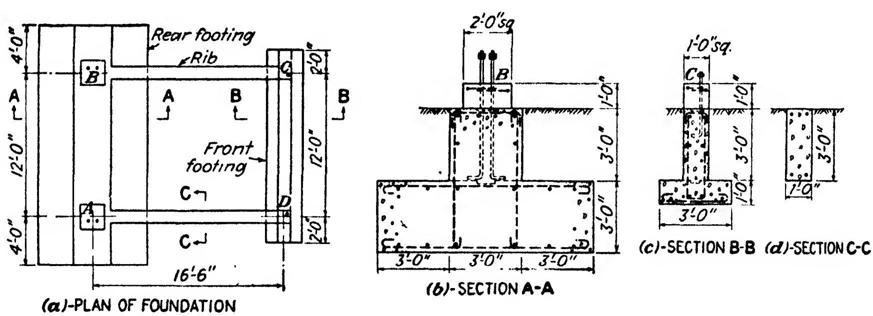


FIG. 8-25. Design for foundation of a dead-end tower at an industrial plant.

smaller cross wall and footing in front, and two ribs to resist the tipping tendency of the footings. Therefore, for practical purposes, the structure may be analyzed as though it tends to tip about rib CD, the leverage of the rear footing is AD, the pressure under the front footing is reasonably uniform, and friction and the resistance of a large volume of earth prevent sliding. The weight of the rear part minus the ribs is 144 kips; the ribs, 13 kips; and the front part (CD) 24 kips. Upon these assumptions, the safety factor against overturning about CD is, taking moments for the entire structure about the bottom and middle of the front footing (a conservative procedure),

$$\text{S.F.} = \frac{144 \times 16.5 + 13 \times 7.75}{70 \times 16.5 + 36.4 \times 5} = 1.86$$

Then, taking moments about AB at the level of the bottom of the front footing, the unit bearing pressure under CD is

$$p_1 = \frac{13 \times 8.75 + (80 + 24)16.5 + 36.4 \times 5}{16.5(3 \times 16)} = 2.5 \text{ k.s.f.}$$

This foundation may be made slightly smaller if the safety factor is only 1.6. It should be noticed that the necessity of holding down the back of

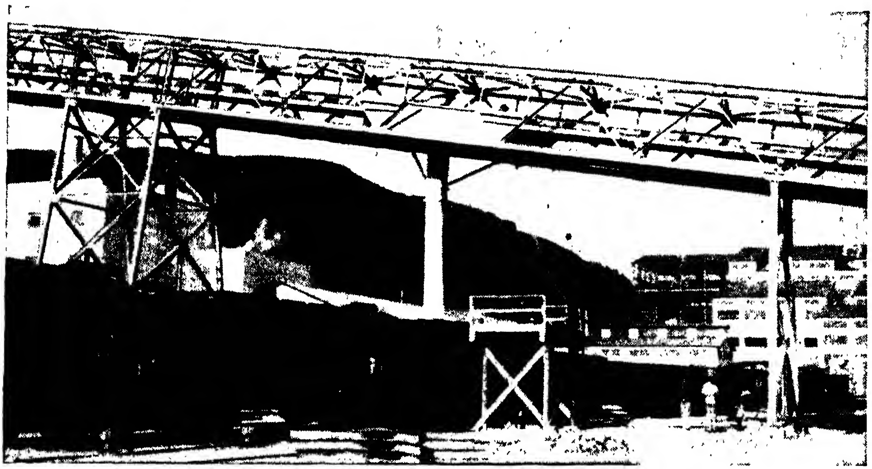


FIG. 8-26. Conveyor trestle under construction at the Morenci Reduction Works, Morenci, Arizona. Notice the single bent and the braced anchor tower. The rectangular chute above the railroad tracks will be used for filling cars. (*Courtesy of the Phelps Dodge Corp.*)

this tower automatically results in such large footings that the bearing pressure upon the ground is small. For such a structure, the wind should be classed as a live load, and the allowable unit stresses should not be increased because of this simultaneous action; the structure is too important.

Comparing the volumes of excavation and concrete, and the practicability of these last two suggested designs, the footing in Fig. 8-24(*b*) seems to be more economical, but one of the general size and shape of that in Fig. 8-25 will be safer and will not depend so largely upon the weight of the earth above it. There have been instances in which a workman has removed the soil on top of a footing while installing pipes, ducts, or an adjacent structure, and he did not suspect that the designer had depended upon that soil remaining undisturbed.

There are many other structures which must resist considerable overturning because of wind, but for which the action of the latter is not so critical. For example, consider the foundation for a single steel bent to support a large elevated conveyor gallery like that shown in Fig. 8-26, assuming good sandy soil.

The bent is intended to support vertical loads and transverse wind loads only. In some cases, where the overturning moment is small or where the legs of the trussing are flared apart to provide large leverage, isolated trapezoidal or spread footings may be adequate and economical. In fact, lateral spreading of the columns to eliminate any net uplift at the windward bearing is basically good planning. When the bent is narrow, however, it is advisable to use a single narrow footing under the pedestals, as shown in Figs. 8-27(a) and (b).

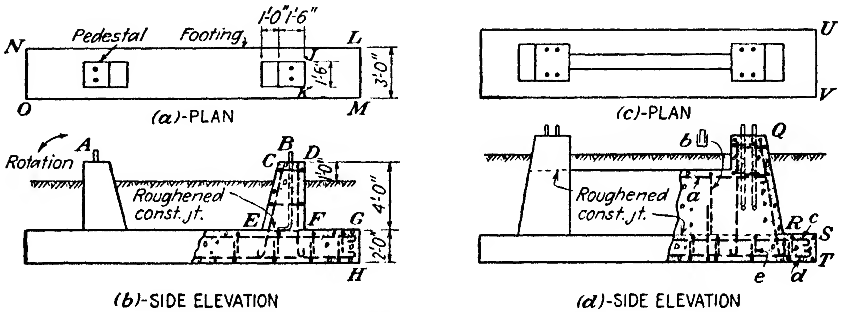


FIG. 8-27. Some footings for a single bent.

These are some of the steps in the design of such a foundation:

1. Compute the maximum and minimum vertical dead and live loads caused by the superstructure.

2. Compute the overturning moment and shear, with the upward and downward reactions at A and B.

3. Select an area of pedestal to fit the bearings.

4. Assume a trial length, width, and thickness of footing.

5. Select the elevation of the top of the pedestals above the ground, and the position of the footing H so as to have it below the frost level, or such that a reasonable weight of soil is on top of the footing. Make the footing symmetrical because of the reversal of wind loads.

6. Proportion the pedestals to resist bending about EF.

7. Test for safety against overturning about H, using the methods illustrated in connection with Fig. 8-24(b).

8. See if the maximum bearing pressure is satisfactory.

9. Design the reinforcement in the footing, the anchor bolts, and the rods to withstand uplift on the windward pedestal.

If greater depth is needed to obtain more weight of earth and if the forces are large, the ribbed design in Figs. 8-27(*c*) and (*d*) may be more desirable.

A feature that may be important in the design of some bents is the participation stress in the diagonals. For example, the structure may be braced as shown in Fig. 8-28, with member *AB* missing. When members *FC* and *GD* are compressed, the diagonals *AG* and *FB* are also compressed and may have a unit stress approximately equal to one-half that in the columns. There is then a horizontal component *H* that may try to spread the pedestals slightly. However, the greater danger lies in the severe bending and shear caused in *CAF* and *DBG*. On the other hand, if *AG* and *FB* are very slender so that they have little value except as tension members, the wind shear may be concentrated at the windward pedestal. Member *AB* should be used as a part of the bracing system. A strut between the pedestals as a part of the foundation will support the pedestals, but it will not relieve the local stresses in the steelwork.

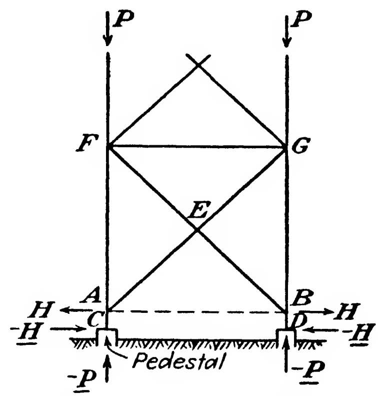


FIG. 8-28. Overturning forces caused by participation stresses.

Long trestles, conveyor galleries, and viaducts that cannot be anchored to abutments or other structures generally must have an anchor tower to resist longitudinal wind as well as transverse forces. The foundation shown

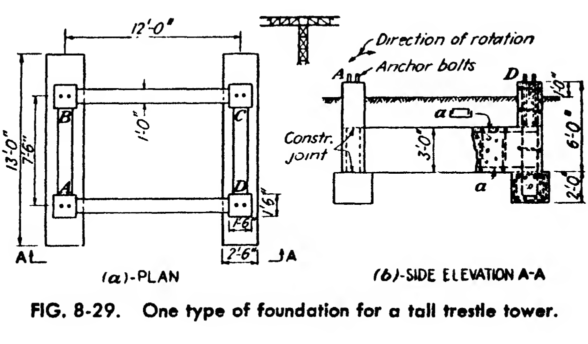


FIG. 8-29. One type of foundation for a tall trestle tower.

in Fig. 8-29 is only one type that may be used when side clearances require the use of a narrow superstructure. This superstructure's trusses are supposed to be parallel to *AD* and *BC*. The substructure should be analyzed separately for transverse, longitudinal, and diagonal wind. For the last, the general method indicated by Eq. (8-8) may be used, although the transfer of "longitudinal" shearing forces between parts of the substructure to bring into action all the bearing areas may be questionable.

Sometimes it is necessary to found bents and anchor towers on the slopes of cuts or on steep hillsides. The bearings may be placed at different levels down the slope, or the substructure may be built to provide level bearings for the steelwork, as shown for one instance in Fig. 8-30. The overhead structure may be parallel or perpendicular to the slope. The center of the bottom of the footing F in (b) should be considered as the point of rotation for downhill tipping, although it may be theoretically possible to use point G. As the supporting power of the earth against downhill shearing at

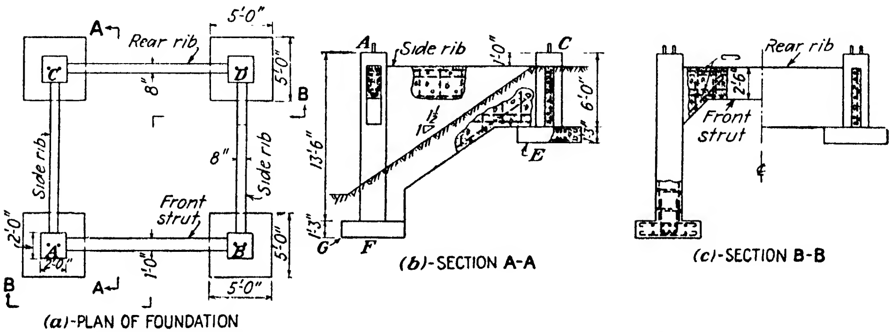


FIG. 8-30. Foundation for a tall conveyor tower at edge of slope or on a hillside.

footing E is questionable, the ribs are made so that all the force may be transmitted to F. For tipping parallel to the hillside, the side AC will rotate about F, whereas BD will try to rotate about E. If the front and rear portions are safe when considered as independent halves "cut" along the center between A and B, the combined structure will probably be safe also. However, it is sometimes desirable to analyze the structure as an entity and to see that it cannot tip about line FE or about either downhill corner.

Example. Figure 8-31(a) shows a steel A-frame that is to be used to support one side of an outdoor crane runway. Assume that $P_1 = 10$ kips DL and 70 kips LL, and that $S = 10$ kips. The soil is deep medium clay that has been covered with 3 ft. of cinder fill. The location is Baltimore, Md. The longitudinal bracing is not connected to this particular frame. Design a suitable foundation for this superstructure, using a safety factor of 2.

The cinder fill is not suitable for the support of a structure like this one where appreciable subsidence, uneven settlement of neighboring footings, and tilting caused by unequal settlement of the ends of any individual footing will affect the alignment of the rails seriously. The footing should therefore be placed well down into the clay. A maximum allowable unit pressure of only 4 k.s.f. will be used in order to avoid serious pumping action and compaction.

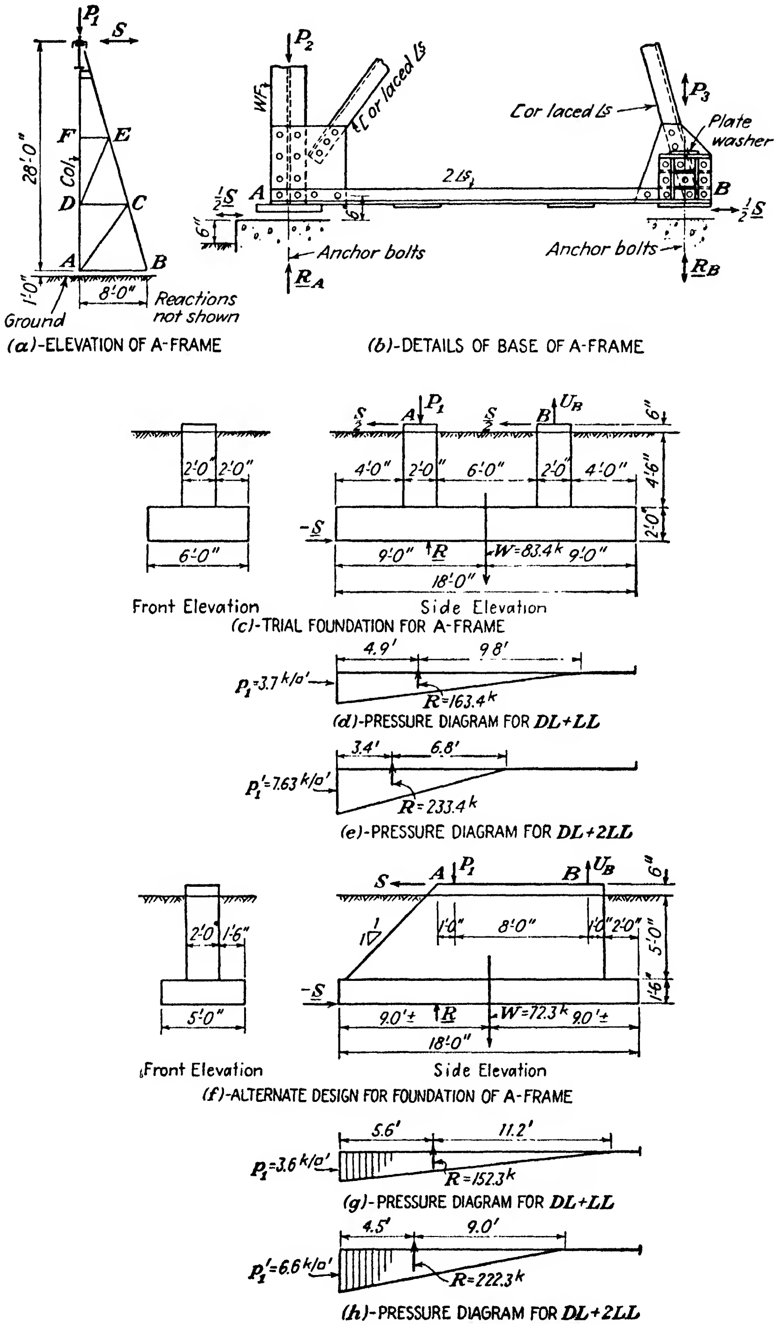


FIG. 8-31. Support for outdoor crane runway.

Since the top of the frame is free to tilt sidewise, the load P_1 should be assumed to act fully upon point A, because participation-stress action will not affect B unless the top cannot move horizontally. The web members and back leg are to brace the front column and obtain anchorage to keep the column from tipping over.

The critical load case is when P_1 is a maximum and S acts toward the left in Fig. 8-31(a). Consider the following designs for the substructure:

1. Try separate footings under A and B. The uplift at B for design purposes is $U_B = 2 \times 10 \times (28/8) = 70$ kips. This requires the equivalent of approximately $70/0.15 = 467$ ft.^{3} of concrete, close to an 8-ft. cube and obviously unreasonable. The downward reaction at A is

$$P_A = 80 + \frac{10 \times 28}{8} = 115 \text{ kips}$$

or

$$2 \times 70 + \frac{2 \times 10 \times 28}{8} + 10 = 220 \text{ kips}$$

when including the safety factor. This footing must then have an area in excess of $115/4 = 29$ ft.^{2}

These footings will practically bump into each other, and no allowance has yet been made for the additional effects of overturning caused by the shear $S/2$ applied at the tops of the pedestals at A and B. It seems, therefore, that a combined footing should be used.

2. Try a foundation like that shown in Fig. 8-31(c). The pressure diagram for the dead load $+P_1 + S$ is shown in (d); that for dead load $+2P_1 + 2S$, in (e). These are computed as though the shear at the top of the substructure is S or $2S$, that it acts upon a solid structure, and that it has a lever arm of 7 ft. above the bearing area. This foundation may be improved.

3. Try the foundation shown in Sketch (f). This provides relatively more bearing area in the vicinity of A and has a continuous rib, and the footing is narrower and thinner. The pressure diagrams for the two load cases are shown in (g) and (h). The rib wall now serves as the backbone of the structure, and the footing is not subjected to serious bending and shear. The longer projection in front of A also reduces the maximum toe pressure.

The third design seems to be the best of the three. It is probably impractical to use a foundation that will have absolutely uniform pressure under it for the load P_1 alone. These loads are temporary and will not produce flowing of the soil unless the pressures are excessive. In a certain industrial plant, a high edge pressure was disregarded. The clay yielded progressively. According to the latest reports, the settlement of the bearing that corresponds to A of Fig. 8-31 (a) amounted to 6 or 8 in. for some of the foundations so that successive adjustments have been made by shimming, welding on extensions to the anchor bolts, and building and rebuilding concrete pads under the base plates in order to line up the crane girders. The results of this faulty design have been both unfortunate and expensive.

8-10. Anchors for backstays. A column and backstay, such as pictured in Fig. 8-32(*a*), constitute a simple and economical structure to resist

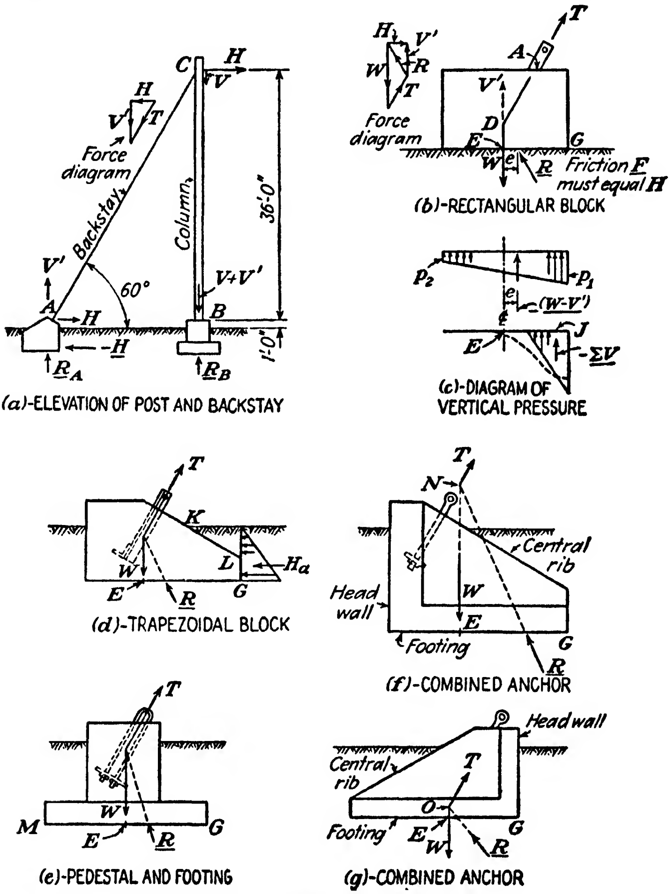


FIG. 8-32. Anchorage for a backstay.

horizontal and inclined forces applied to its top. In some cases, to secure lateral stability, it may be desirable to use two columns that are sloped to form an A-frame; in others, to use a single column and two backstays that are splayed at an angle of 60 to 90°, or more.

For simplicity of illustration, the structure shown is assumed to consist of a single column *BC*, that is steadied laterally in a plane perpendicular to the drawing, and a single wire-rope backstay AC. When a force is applied at C with components H and V, the former must be resisted by the backstay because the column is practically free to rotate about its base. The tension in the backstay is therefore T, as shown by the force diagram, and the forces applied to the foundations by the frame are as indicated, the dead load of the structure being neglected.

It is obvious that the footing under B may be designed for vertical loads only. When BC is an A-frame, this foundation should be designed to resist the vertical loads and whatever transverse forces may exist. The A-frame may be considered to constitute a single bent, and it generally is braced to act as one.

The forces applied to the foundation at A try to pull it out of the ground, to drag it toward B, and to tip it over. The action of a simple gravity anchor—one dependent upon its weight alone—is pictured in Fig. 8-32(*b*). The horizontal component of T is resisted by the friction F, and these forces constitute a couple $H \times DE$ that tends to tip the block about G. The force W is the weight of the block treated as a single force at the center of gravity, then $W\text{-}V'$ is the vertical component of the resultant R. The righting moment that resists tipping is $(W\text{-}V')EG$, unless the pressure under G is so large that possible yielding of the earth makes it advisable to assume a point of rotation closer to E. If the forces are as shown to scale in Sketch (*b*), the pressure diagram may be assumed to be as pictured in the upper sketch in (*c*). The safety factor against uplift is W/V'; against overturning, $W \times EG/(V' \times EG + H \times DE)$; against sliding, $Wf/(V'f + H)$, where f is the allowable coefficient of friction, or friction plus cohesion. However, the last two may be deceptive. It is advisable to increase T by the required safety factor, then to compute the maximum pressure at G. When the latter is excessive and the pressure diagram is supposedly like that pictured by the solid lines in the lower sketch in (*c*), the pressure diagram may be more nearly like that shown by the dotted lines, and the actual point of rotation may be at some uncertain point J. Rather than try to locate J theoretically, one might better modify the design to ensure safety.

In such anchorages as these that are not on rock, it is usually desirable for reasons of economy to sink the anchorage into the ground so that the abutting power of the soil in front of the foundation can be utilized to assist in the resistance to sliding and overturning. For analysis, one may use the methods explained in the previous article. However, when testing for the desired safety factor, $T' = T(\text{S.F.})$ should be used, and the analysis made accordingly. In fact, if one assumes T' in the first place and then designs a foundation that will not quite fail under those conditions, he will know that his structure is safe, and he need not care what the pressures and stresses may be under operating conditions.

Many are the varieties of anchor that may be used to hold a backstay. Some are shown in Fig. 8-32. For the correspondingly labeled sketches the following comments are given:

(*d*) This is designed to shift W toward the rear, to increase the leverage EG, to equalize the pressure on the soil somewhat, and to utilize the resistance of the earth H_a. The soil above KL produces useful weight, but the possibility of its sliding along KL may cause doubts as to the reliability of that part of the soil above L in producing lateral resistance. The friction of soil on smooth concrete may be less than that of earth on earth.

(*e*) This design shows an attempt to minimize the quantity of concrete, and to utilize the weight of earth above MG as a part of W. This plan is not likely to be desirable for large pulls.

(*f*) This is an L-shaped concrete slab with a central stiffening rib. It attempts to utilize a large volume of earth for weight and to have considerable breadth to bear horizontally against the soil. The height of point N may cause large eccentricity of the resultant at the base.

(*g*) Here the structure in (*f*) is practically reversed. The advantage gained is the low position of point O and the decrease in the eccentricity of R.

In connection with all these anchorages shown in Figs. 8-32(*d*), (*e*), (*f*), and (*g*), notice the following:

1. The wire rope backstay is assumed to have a clevis at its lower end. Between the clevis and the steelwork of the anchorage there should be a turnbuckle or some other device to permit adjustability of the backstay.
2. The steelwork of the anchorage should be thick so as to reduce the danger from rusting at its junction with the concrete.
3. The steel should terminate in such a manner that it engages the concrete mechanically rather than through bond alone.
4. The concrete should be reinforced so that it cannot fail as a whole or in detail.

When bedrock is at or near the surface, a backstay may be anchored directly to it. At first glance, it might seem that the anchorage shown in Fig. 8-33(*a*) is ideal. To be considered, however, is the question of how the inclined hole can be drilled. If the eyebolt AB with an upset end at B is to be grouted or concreted in place thoroughly, the diameter of the hole should be so large that it will be difficult to make it with standard pneumatic equipment. The vertical hole shown in (*c*) is therefore more practicable because it can be made with a churn drill. The anchor should be bent to a long-radius curve at F where it is embedded in the concrete. The rock near G should be broken off or roughened to provide adequate shearing resistance so that the block J will not slip on the rock.

The following comments also apply to the construction shown in Fig. 8-33:

1. *Instead of the upset end B, the Fox bolt arrangement in (b)* may be used. Unless the wedge *E* is welded to *D*, failure of the bond along the rod may cause the latter to pull away from the wedge. A large welded bead around the end of the rod may serve as an even better anchor.

2. The threaded end *H*, if long enough, will be an excellent anchorage because of the high strength in bond.

3. It may be cheaper to thread the rod and put on a standard clevis or other fitting *E* than to forge and drill the rod as at *A*.

4. The rock must be able to resist the pull without cracking and spalling.

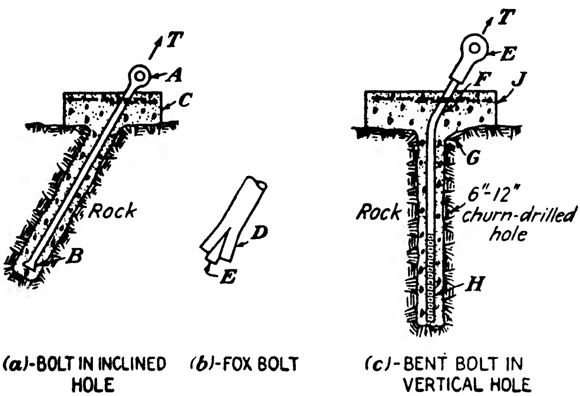


FIG. 8-33. Anchorage in rock for backstay.

5. The drilled hole should be deep enough to provide a reliable anchorage. The top foot of the hole may have little value because of local weakness of the rock.

6. The concrete pad around the top of the anchor rod in (*a*) is useful principally to make a workmanlike job, to protect the steel near the rock line, and to prevent serious bending in the rod due to any cause.

When planning the structural connection between a foundation and a member that is subjected to violent and sudden uplift, one should remember that long highly stressed anchor bolts or steel ties will deform appreciably. If anchor rods 10 ft. long connect the end of such a member to a buried beam or girder near the bottom of the concrete foundation and if these rods are stressed an average of 15,000 p.s.i., the elongation may be approximately $\frac{1}{16}$ in. Under shocks and repeated or reversible loads, as in a large headframe like that in Fig. 8-55, this may cause objectionable vibration and "chattering" of the superstructure even if it does not crack the concrete. Prestressing of the rods may be desirable.

8-11. Pedestals on large mats. It is possible to have a condition in which a group or series of structural parts or machines may be founded upon a large common mat. One, several, or all parts may be loaded at a given time, and they may cause overturning moments and shears as well as vertical forces. A common mat may have been chosen because of the low bearing value of the soil, the desire to tie all parts together as a unit, the advisability of spreading the effects of any local and variable loads, and the need of a concrete floor over the area so that it is economical to incorporate this floor in the substructure itself.

When critical cases are encountered, careful analysis of a tentative plan may be necessary. However, empirical design is generally sufficient for most practicable structures as far as overturning effects are concerned. If the use of a mat entails serious uncertainties, one might better change the basic plan to some more trustworthy and determinable arrangement.

Assume for illustration that Fig. 8-34(*a*) is the plan of the substructure for a battery of tall vertical cylindrical steel tanks 5 ft. in diameter, which may be full or empty at any given time. Assume further that they are part of the equipment of an industrial plant in a warm climate where there is no danger from frost and that they may be subjected to hurricane winds.

Each tank is on a base or pedestal of the dimensions shown in (*a*) and (*b*). For supporting vertical loads, the mat is to have two longitudinal and four transverse reinforced strips 10 ft. wide. These strips may be proportioned to spread the localized loads as already explained in Chap. 7. However, if the loading on each longitudinal strip is as shown in Sketch (*b*), what will be the effect of the overturning forces upon the mat?

In Fig. 8-34(*c*) is shown an exaggerated picture of the deformations that might occur because of the moments M and shears S of Sketch (*b*). As each tank tries to tip toward the right, the end A of the mat will tend to lift up, causing little restraint except for its dead-load moment. End D will tend to act like one side of a footing subjected to overturning. Between E and H the adjacent tanks cause bending and transverse shears in the mat, and the uplift at R is restrained through the mat by the compression at Q. The shear caused by F between QR is pictured by $-V_{t_1}$; that caused by E, by V_{t_1}. Somewhere between adjacent pedestals there will be a point of inflection in the mat.

The bottom reinforcement under Q should be proportioned to resist all of M_e; the top steel at R and the bottom steel under T, for $\frac{1}{2}M_f$; the top at U and bottom under W, for $\frac{1}{2}M_g$; and the top rods at X and bottom below Y for $\frac{1}{2}M_h$. Any bending caused by the shears S should be resisted similarly. Of course, the mat should be reinforced for wind from the other directions also.

The preceding recommendation is based upon the assumption that the mat is stiff enough to prevent distortions that would affect greatly the bearing on the soil. The rotations might cause some redistribution of pressure as indicated by Fig. 8-34(*d*) but, if the mat is made strong enough to withstand the moments by itself, the structure will not fail, and moderate variations of the pressure upon the soil need not cause worry.

If the pedestals are strong enough, the critical bending will occur at their bottom corners; otherwise, the greatest bending will probably be near the

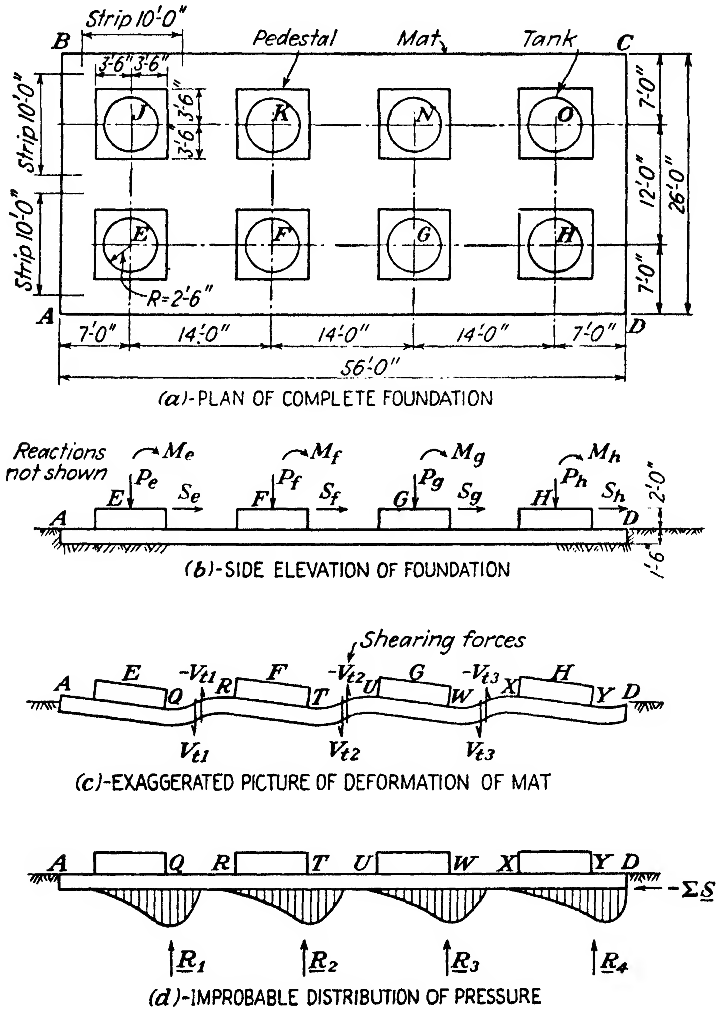


FIG. 8-34. Overturning action of a series of pedestals on a large mat.

leeward side of the tanks and near the windward anchor bolts. Transverse reinforcement is also needed to spread the effects of localized forces.

Some further pictures of the suggested construction for the preceding foundation are shown in Fig. 8-35. Notice the following.

1. The vertical load will cause tension in the bottom steel of the mat, and the rods are extended clear across under the pedestals.

2. The anchor bolts should be embedded thoroughly. If they are short like g in Sketch (c), the bond stresses on the bolts and reinforcement may be too large.

3. The top rods, like b and d in the mat, are extended clear across the pedestal to obtain proper anchorage for tension both at R and at T.

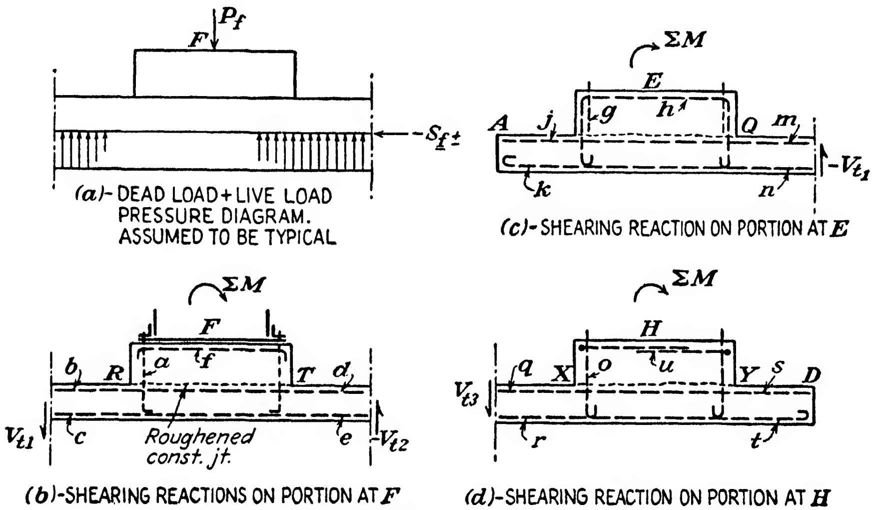


FIG. 8-35. Assumed forces on mat because of overturning.

4. The pedestal is poured after the mat has set. The rods f are merely to knit the top together, whereas h does this and also ties down the pedestal.

5. The shearing stresses in the mat may be assumed to be equal to $\Sigma M/L$, where L is the spacing of the pedestals except in the case of E, the one on the windward side. Here the shear near Q may be considerably higher, perhaps by 50 per cent.

6. The transverse reinforcement is not shown in detail. It may be determined in the same general manner as is the longitudinal steel.

7. In general, it is advisable to determine the required top and bottom reinforcement of the arbitrarily chosen strips—10 ft. wide in Fig. 8-34(a)—for dead load and live load; test for the combination with overturning, allowing 1.3 times the permissible unit stresses; increase the reinforcement of the mat to resist the combined loads if necessary; and then extend the reinforcement clear across the mat, except for minor local additions that may be needed in the bottom under the end pedestals, such as E and H.

One cannot expect to illustrate all possible problems that a designer may encounter. However, this problem shows how one may assume a definite system in an indeterminate mat, then make this system able to resist the applied loads, relying upon the probability that the structure will not fail if there is a way in which it can stand up. On the other hand, the choice of such a system should be made with good judgment and with reasonable probability that the structure can act as intended.

When a single local overturning moment of large magnitude is applied in an intermediate position upon a large mat, or when numerous such moments are applied too far apart to cause trustworthy counteracting resistance in the mat as illustrated in Fig. 8-34, then it is probably advisable to design the mat near the load so that it will be strong enough to act somewhat like an isolated spread footing. A strong pedestal on top or a suitable thickening below, as in Fig. 7-4(*b*), may be satisfactory. Even the construction of a depressed rib, as in Fig. 7-7(*a*), to form a stiff beam in the desired position may be advantageous. In any case, one should be sure that the substructure cannot tear apart locally near the edge of the structure that applies the loads and that the concrete will not fail there by shearing.

8-12. Retaining walls. A brief discussion of the design of retaining walls is necessarily a part of this chapter, but more complete data are published elsewhere.[1]

In Fig. 8-36(*a*) is shown a T-shaped reinforced-concrete retaining wall that is to hold back a sloping embankment. The lateral pressure of the earth and of any water impounded behind the wall tend to tip it over about A, to slide it along the soil under the footing AB, and to break its individual parts as indicated in (*c*). Referring to the various sketches in Fig. 8-36, the design procedure may be described briefly as follows:

1. Choose a height and shape of wall to meet the general requirements of foundation, property lines, and any other local conditions that influence the problem. The width of the footing L should be approximately one-half the height EJ of the wall, the thicknesses of parts should be substantial, and AB should be below frost depth.

2. For stability and resistance to sliding, ΣW equals the weight of the concrete and of the earth above the footing AB, applied at the center of gravity x_o.

3. According to Coulomb's formula, the lateral thrust of the soil may be assumed to be

$$P_e = \tfrac{1}{2}wh_1^2 \frac{\cos\phi}{(1+\sqrt{2\sin^2\phi - 2\sin\phi\cos\phi\tan\delta})^2} \qquad (8\text{-}12)$$

[1] Clarence W. Dunham, "The Theory and Practice of Reinforced Concrete," Chap. 8, McGraw-Hill Book Company, Inc., New York, 1944.

where the various terms are as shown in (*a*), and w is the unit weight of the soil. When $\phi = 30°$, $\delta = 0$, $w = 100$ p.c.f., and h_1 = the depth from the surface of the ground to the point considered, Eq. (8-12) becomes

$$P_e = 15h_1^2 \tag{8-13}$$

The direction of the thrust is assumed to be inclined downward at the angle of repose or of internal friction of the soil, and the distribution of the pres-

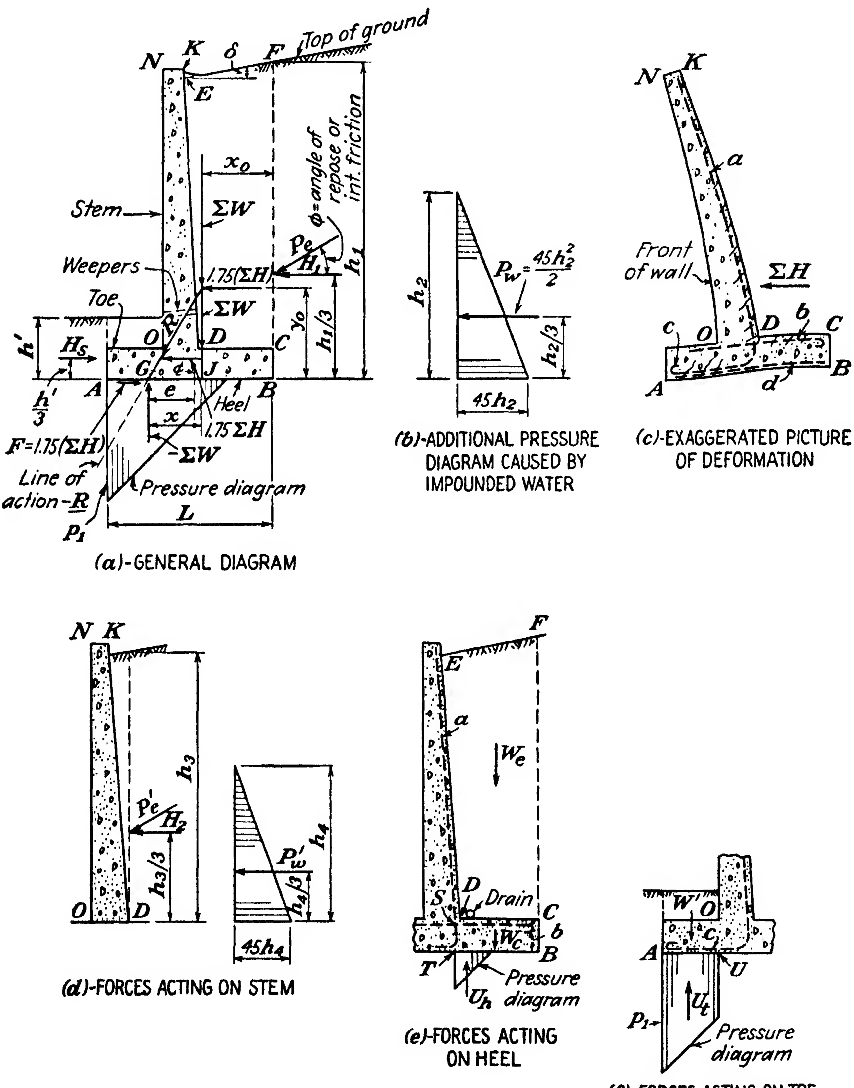


FIG. 8-36. Data for the analysis of a reinforced-concrete retaining wall.

sure is assumed to be triangular or hydrostatic. If the embankment supports a roadway close to the wall, the vibration makes the magnitude and direction of P_e uncertain, and it is more conservative to assume the full value of P_e acting horizontally.

4. When the wall rests upon rock or fairly impervious soil, water may collect behind it unless removed by longitudinal drains or weepers. The additional thrust caused by water pressure may be assumed as equivalent to the hydrostatic pressure caused by a liquid with a unit weight of 45 p.c.f., as shown in (*b*).

5. The total horizontal force, neglecting surcharge and frost action, is $\Sigma H = H_1 + P_w$, and it is applied as their resultant at y_o.

6. The safety of the structure may be tested by multiplying ΣH by the safety factor, usually 1.5 to 2.0, and analyzing the wall for this condition. For illustrative purposes, a safety factor of 1.75 is used, which is satisfactory for most purposes. For these ultimate conditions, the yield point stress of steel, $1.75 \times f_c$ for concrete, the ultimate compressive resistance of the soil, and the maximum coefficient of sliding friction f may be used as limiting values.

7. For testing the stability against overturning, take moments about A, or a point somewhat behind it if the pressure at the toe is too large. Then the righting moment should at least equal the overturning moment, or

$$\Sigma W(L - x_o) \geq 1.75(\Sigma H)y_o \tag{8-14}$$

8. For testing the stability against sliding, the allowable frictional resistance should at least equal the active horizontal force, or

$$(\Sigma W)f \geq 1.75(\Sigma H) \tag{8-15}$$

When a large wall rests upon plastic soil, the designer may have difficulty in providing sufficient resistance to sliding. In some cases, he may include with the friction the abutting power H_s of the earth in front of the wall, as shown in Sketch (*a*), assuming

$$H_s = \frac{wh'^2}{2}\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right) \tag{8-16}$$

as computed from the Rankine formula, Eq. (8-10). However, this may be dangerous if someone excavates in front of the wall.

9. The eccentricity of the resultant R may be found as indicated in Sketch (*a*) and as explained in Art. 8-6. The soil pressures and the pressure diagram may then be determined by means of Eq. (8-4) or (8-6).

10. The maximum bending and shear in the stem of the wall are at OD, Sketch (*d*). Neglecting the weight of the stem and multiplying the forces by the safety factor,

$$M_{OD} = 1.75\left(H_2\frac{h_3}{3} + P'_w\frac{h_4}{3}\right) \tag{8-17}$$

and

$$H_{OD} = 1.75(H_2 + P'_w) \tag{8-18}$$

11. The critical section of the heel is at S, Sketch (e), where rods a and b cross each other. The maximum bending at S for 1.75 times the horizontal forces is

$$M_s = W_e \times \text{lever arm} + W_c \times \text{lever arm} - U_h \times \text{lever arm} \tag{8-19}$$

The shear at S is

$$V_s = W_e + W_c - U_h \tag{8-20}$$

These two formulas are left in a generalized form because the shapes and magnitudes of the earth mass W_e, the heel W_c, and the pressure diagram U_h vary with specific cases. It is also desirable to analyze the heel for the condition using the ordinary computed values of the horizontal forces, the accompanying pressure diagram, and customary working stresses because this case may control the design.

12. The critical bending in the toe will be at OU of Sketch (f). Using symbols similar to those in item 11,

$$M_o = U_t \times \text{lever arm} - W'_e \times \text{lever arm} - W' \times \text{lever arm} \tag{8-21}$$

and

$$V_o = U_t - W'_e - W' \tag{8-22}$$

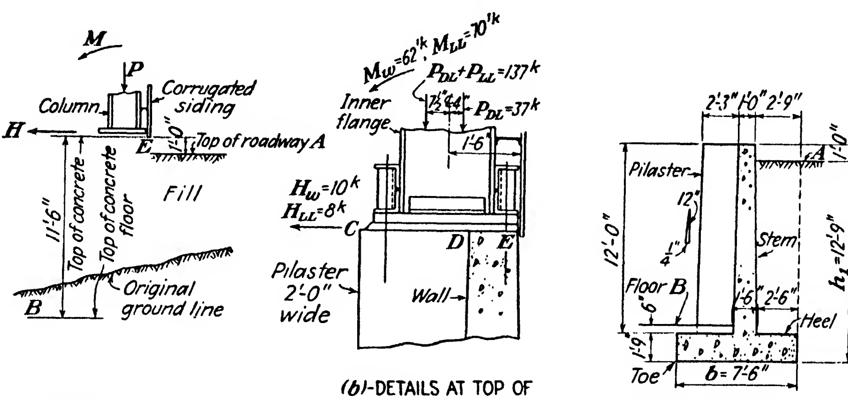
This part of the structure should also be analyzed to see that it is safe for the ordinary forces and unit stresses.

Walls of many types and shapes may be used. Construction joints, contraction joints, paneling for architectural appearance, batter of the front for aesthetic reasons, drainage, cutoff walls under the footings, sloped or stepped copings and footings, waterproofing, and surface finish—all these are important, as in the case of basement and foundation walls.

8-13. Foundation walls with lateral loads. A wall may be in part a foundation, in part a retaining wall. Lateral loads may be caused by wind, cranes, earth, frame action, and many other things; they may be applied by the superstructure along with its vertical loads, or independently. The longitudinal bending produced by concentrated loads may be approximated as explained for foundation walls; the lateral bending, as described in the preceding article provided the additional vertical and horizontal forces are included in the calculations. Exactness of computed pressures and unit stresses is not to be expected, nor is it usually essential. However, experience shows that safe and reasonable structures can be built, and that sufficiently accurate approximations for design purposes can be made, if sound judgment is used in the process.

It is obvious that one should plan a structure to minimize uncertainties and dangers. Not always is the construction used by our predecessors to be followed blindly, neither is it to be discarded thoughtlessly in favor of an untried design. One of many arrangements might be used for a substructure; the one should be used which is most trustworthy, yet practicable and economical.

As a general illustration of combined action, assume the condition shown in Fig. 8-37(*a*). This represents one side of a large steel mill building to be located on sloping ground. The area *A* is to be filled in and used as a plant roadway between this building and another one farther up the hill.



(*a*)-GENERAL REQUIREMENTS (*b*)-DETAILS AT TOP OF WALL AND PILASTER (*c*)-TRIAL DIMENSIONS OF WALL

FIG. 8-37. Combined foundation and retaining wall for a mill building.

The soil is a clayey sand that has a safe bearing value of 6 k.s.f., and the area has been used as second-growth woodland. Each column is to be a 24-in. wide-flange section with the crane girder seated on the inner flange, and with a 10-in. column spliced on top of the other portion to support the roof. These columns are to be 20 ft. c.c. What type of construction is desirable?

The following outline may illustrate one solution for the problem:

1. A continuous wall is needed to hold back the ground.

2. The columns are so deep that pilasters are needed under them, since the top of the wall need not be thick enough throughout to support the columns directly.

3. The bottom of the corrugated siding is at *E*. It is desirable, therefore, to place the top of the wall so as to form a continuous line at *E*, Fig. 8-37(*b*), with the pilaster projecting inward a distance *CD*. This provides more space within the building than would be the case if the wall lined up with *C*, and it avoids the widened top that would be necessary to close across to *E*.

4. The wall must serve as a combined foundation and retaining wall. Therefore, a T-type will be used, the width of footing b will be made a little wider than $\frac{1}{2}h_1$ of Sketch (c), the inside of the wall and pilaster will be battered $\frac{1}{4}$ in. per ft. to avoid the appearance of leaning inward, and the toe will project a little beyond the base of the pilaster. This causes the column load to be rather close to the toe so that large toe pressure may cause an increase in the width of the footing.

5. The trial dimensions of the wall are shown in Sketch (c).

6. The footing will be placed below the floor slab. This permits the installation of the floor after the heavy construction work is completed, and the floor can then be finished and sloped for drainage as desired. This floor, being wide and heavy, may be relied upon to assist in the resistance to sliding of the wall if necessary.

7. The pavement A will be a flexible bituminous type because of economy and the fact that much of it is on fill.

The loading conditions for which the wall should be analyzed are the following:

1. Temporary safety as a retaining wall with the fill in place, the floor B omitted, and the weight of the superstructure not applied.

2. Safety of the completed structure with the dead load of the superstructure combined with maximum H_w and M_w caused by wind. In Sketch (b), the position of this dead-load reaction P_{DL} is shown.

3. Safety of the completed structure with the dead load and the maximum live load combined with the possibly coincident live-load shear H_{LL} and overturning moment M_{LL}. The new position of the vertical reaction is indicated in (b). There may be cases also in which the omission of live load upon the roof causes more critical conditions.

4. Safety of the completed structure with dead load, maximum live load, and wind all acting simultaneously. A 30 per cent increase in the allowable unit stresses is then permissible.

5. Safety of the completed structure with dead load, minimum vertical crane live load, maximum H_{LL} caused by the crane at the wall when the trolley is at the other end of the bridge, and the corresponding M_{LL}.

6. The safety factor may be applied in the live and lateral loads as illustrated in the preceding article when testing for safety.

By an inspection of the forces, it is generally possible to pick from the load cases previously listed the one or two that will cause the greatest effects upon the substructure. If the wall can withstand these conditions, it can resist the less serious ones more easily, and the latter need not be investigated individually except when the results cannot be predicted with reasonable certainty.

The vertical loads from the superstructure may be assumed to cause vertical bending in the stem as a beam, but this may not be serious when the wall is so deep. The wall may generally be analyzed as though it were a huge T-beam. Extra longitudinal reinforcement in the top of the stem is all that is required in most cases. When the wall is shallow, compression in the top near the pilasters, caused by longitudinal bending, may be important.

The horizontal loads from the superstructure may be assumed to be resisted by a considerable length of wall even though they are applied at the pilasters. It is true that the stem is much stiffer laterally at the pilaster than at some vertical section beyond it, nevertheless the monolithic construction will tend to act as a unit unless it cracks. The magnitude of the torsional stiffness of such a wall is uncertain, but the following assumptions are suggested regarding how far along a wall a local concentrated horizontal shear or overturning moment applied at a pilaster may be assumed to spread, as far as the stability of the wall and the strength of its parts are concerned:

1. One-half bay each side of the center of the concentration; 20 ft. in this case.
2. The height of the stem each side of the center of the concentration; 24 ft. in this case.
3. Eight times the thickness of the top of the wall, DE in Fig. 8-37(b), each side of the edge of the pilaster, or the center of the concentration if no pilaster is used. This gives 18 ft. in this case.

The minimum distance computed from any one of these three gives the limiting spread; 18 ft. here. Then the concentrations applied by the superstructure may be divided by 18, and a typical 1-ft. slice of wall may be analyzed for these loads combined with those of the earth and other forces that are assumed to be acting uniformly along the length of the wall. It is obvious that horizontal wind and live loads that act toward the earth behind the wall will not be important in ordinary cases.

For example, assume that Fig. 8-38(a) shows one load case for which a typical section of the wall in Fig. 8-37(c) is to be tested for stability and soil pressure. The moment M may be replaced by shifting the load P to the left a distance $e' = M/P$. The earth pressure P_e per foot of wall will be computed from Eq. (8-13) and it will be assumed to act horizontally because of the vibrations caused by traffic on the roadway. Surcharge is not to be included in this problem. The resultant of H and P_e is ΣH; that of the dead load of the wall and earth W and P is ΣV. With these, the eccentricity e and the ordinates of the pressure diagram may be calculated. The results are shown in Sketch (b). Taking moments about F, the safety factor against overturning seems to be

$$\text{S.F.} = \frac{14.7 \times 3.5}{2.89 \times 5.73} = 3.1$$

These results and the pressure diagram in Fig. 8-38(*b*) indicate that the proposed wall is perhaps unnecessarily conservative for the load case tested. It may not be so if impounded water, wind, and the surcharge caused by a truck are included in the loading.

By no means is the concrete of the pilaster in Fig. 8-37(*c*) negligible as far as stresses in the concrete and reinforcement are concerned. Nevertheless, the effect is somewhat localized, and it will probably stiffen the stem somewhat. However, the pilaster cannot act as the stem of a T-beam that

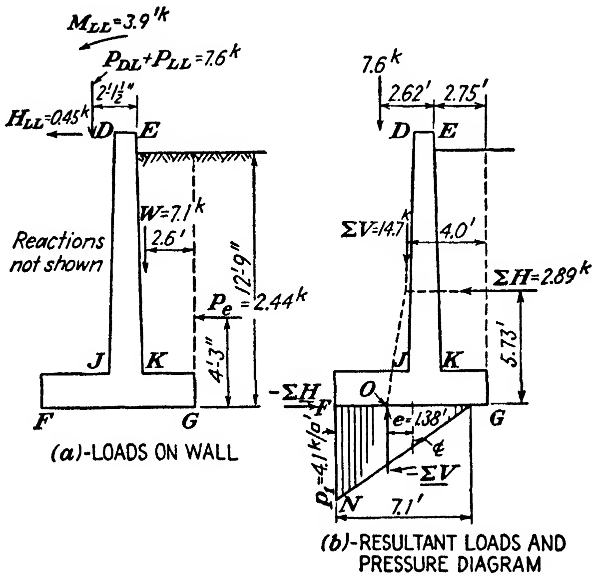


FIG. 8-38. Analysis of wall for one load case.

resists all the overturning because the footing near the pilaster and the soil under it cannot offer the necessary resistance, whereas rock might do so. In planning a foundation, one should remember that he cannot push on something any harder than the latter can react.

Another possible arrangement, instead of that used in Fig. 8-37(*c*), is the moving of the stem of the wall toward the fill sufficiently to clear the column. The latter may then be extended down to the footing. This may be suitable when the footing is on rock or very strong gravel. Otherwise, the moments and shears at the top of the wall may be less troublesome than the heavy concentration that tends to break the toe of the footing locally.

Assume that a wall must be built along a rock cut, as shown in Fig. 8-39(*a*). To minimize excavation, a heelless wall may be used that cannot be waterproofed properly, and this is a situation in which water may be troublesome. The design of the wall should be made with the inclusion of earth pressure because of probable overbreakage. This wall must be thick in order to secure the necessary weight unless P_{DL} is large. Gripping of the rock at the rear of the footing of the wall to resist overturning is unreliable unless holes are barred or chipped in the rock. A better design is the extension of the excavation and heel about 2 ft. as shown in (*b*). This provides space for waterproofing the wall, for the installation of a drainage system, and for the addition of some weight of earth on the heel.

If a rock excavation has been made and looks like that in Fig. 8-39(*a*), or if line drilling and careful blasting can be trusted to produce such results, is it safe to pour the concrete against the rock as shown in (*c*)? Should

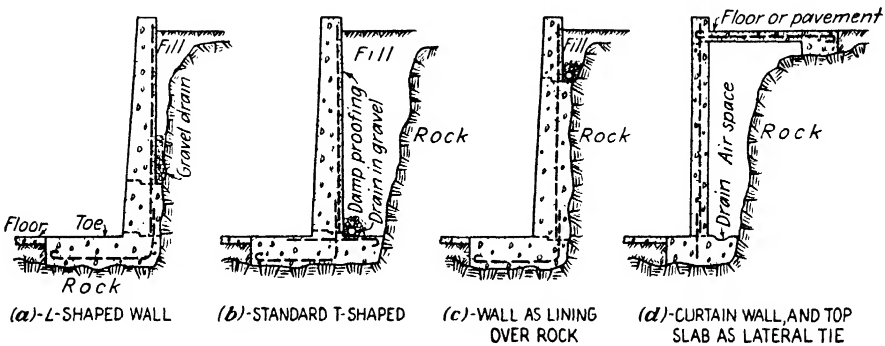


FIG. 8-39. Possible construction in rock excavation.

one assume that there will be little or no lateral pressure against the concrete? Here are some comments about such construction:

1. Ground water will almost inevitably pass downhill through seams in the rock. When the wall is long, it tends to act like a dam so that large hydrostatic pressure may exist.

2. Leakage through such a wall is almost certain because the keying of the concrete to the irregularities of the rock will result automatically in shrinkage cracks. Admixtures in the concrete may improve its density but not eliminate these cracks, and waterproofing applied to the inner surface of the wall is likely to be costly and unreliable.

3. Surface water will almost surely penetrate behind the wall, automatically becoming ground water.

4. Freezing of the ground water, if freezing can occur behind the wall, will force the wall to yield or even to crack badly.

5. Longitudinal drains, or local vertical drains passing under the floor and discharging at the downhill side, may not be completely effective, although helpful.

6. When waterproofing is essential, it may be best to put up a "sand wall," apply a membrane on it, and then make the structure inside—the

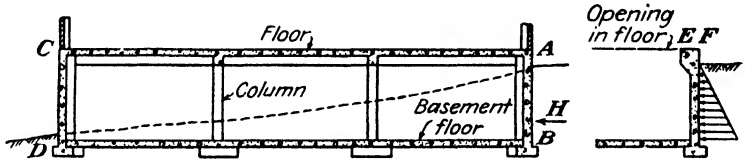
left side in Fig. 8-39(*a*)—capable of resisting the water pressure as well as the horizontal and other loads applied by the superstructure.

7. Drilled holes in the rock with grouted dowels therein to anchor the concrete to the rock may not be helpful permanently. The steel may rust off, and any pressures from ice are too great to be resisted.

8. A construction like that in Fig. 8-39(*d*) may be used if the excavation is not too irregular and if a reliable, inspectable, and cleanable drainage system can be installed.

9. Unless a high-level yard is needed alongside, it may be advisable to excavate the rock to a line farther back so as to eliminate the need for the wall and to obtain better light and ventilation in the structure. The areaway should be wide enough to prevent falling pieces of rock from entering the structure; a continuous concrete wall 2 or 3 ft. high as a base for the side wall may be useful also for this purpose.

There are many cases in which a structure may be designed as a box. For example, assume the basement shown in Fig. 8-40(*a*). The wall at



(*a*)-CROSS SECTION THROUGH A BASEMENT (*b*)-SECTION AT A LARGE HATCHWAY

FIG. 8-40. Illustration of boxlike action of a substructure.

the uphill side will be like a slab spanning vertically between *A* and *B*. The floor *CA*, particularly, may be a strong horizontal diaphragm that will transmit horizontal forces to the end walls. When there is a large opening in the floor *AC* alongside the wall *AB*, the latter may be built with a top that will serve as a horizontal beam, as pictured by *EF* in (*b*). This beam will then support the top of the wall and transmit its end reactions into the adjoining floor. This principle of planning a structure as an entity that is inherently strong and stable should be utilized when practical.

To illustrate a planning problem, assume the general construction shown in the isometric view in Fig. 8-40*A*. This is modeled after a real case that occurred at an industrial plant. Assume that the column loads shown where the steel H's are pictured are 300 kips each; the floor loads, 200 p.s.f. The walls and floors are of reinforced-concrete construction. Plan the construction, assuming the soil to be firm sandy gravel that can support safely a bearing of 4 tons per ft.² The original ground under *DE* is at El. 2088 and slopes upward toward *J* where it is at El. 2096.

Using the 24-ft. wall from *E* to *F* as a specific case, how might this be planned? Here are a few alternates for consideration:

1. Make it a heavy retaining wall with the columns supported upon pilasters projecting from its back face. Fill in behind it to hold the raised floor.

2. Make it a curtain wall with pilasters for the columns, supported laterally by the upper and lower floors. The earth backfill is to be sloped from F back toward J so that the raised floor must be reinforced-concrete beam-and-slab construction supported upon local piers and the edge walls. Consider the form work for the floor.

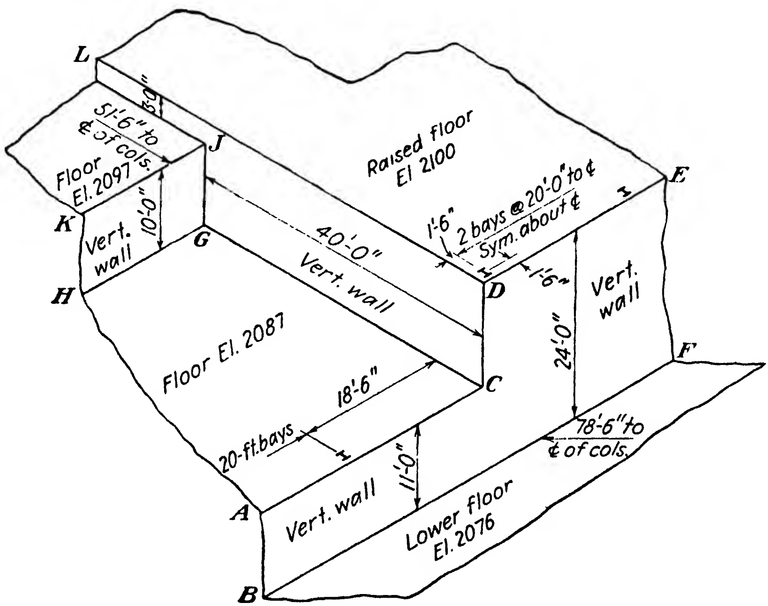


FIG. 8-40A. Problem in planning concrete construction.

3. Make it a strong vertical slab supported laterally by the lower and upper floors, tying the latter together by reinforcement that extends clear back to the column bases beyond L. The upper floor is then placed on backfill.

4. Make it a slab spanning horizontally between ribs or walls at the columns and at the middle of the bays. These ribs are to be triangular, and their bottoms are to slope upward from the footings (or be made in steps) toward J. They serve as a series of counterforts to support the front wall. Fill is to be placed between them, and the floor is to span from the outer wall DJ across these ribs.

5. Make it, the side wall from GJ to CD, the opposite one beyond EF, and the upper floor like an inverted box that is inherently stable. The bottoms of the walls will be supported laterally by the various lower floors. Support the upper floor on fill.

6. Since the upper floor is only two bays wide, a central counterfort might be built under the middle column with a keyed contraction joint on one side of it to cut DE in two. Then one corner portion could be made as an L-shaped structure that is inherently stable because the wing JD serves as a counterfort for the front whereas the front does likewise for the side. The rear part will be U-shaped. The small wall for the intermediate floor will have a joint between itself and the raised portion.

It is obvious that there are many ways to make this structure. Choose the system that seems to be the most practical, then develop it. The

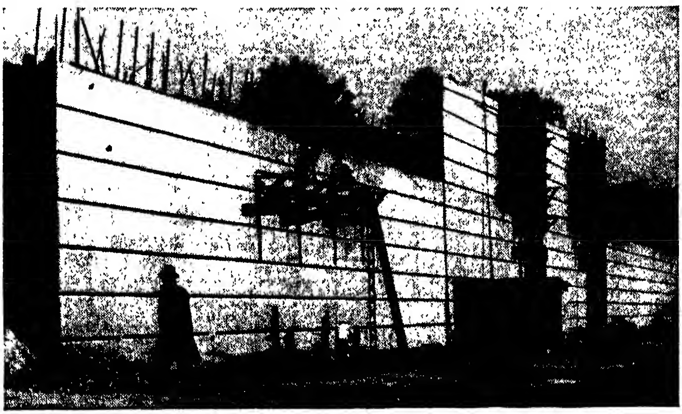


FIG. 8-40B. Retaining wall nearly 50 ft. high along approach ramp to new Rankin Bridge over the Monongahela River, Pittsburgh, Pa. Notice the grooving, the contraction joints, and the sequence of pours. *(Courtesy of Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y. and Department of Public Works, Allegheny County, Pa.)*

plans for the other parts should be selected along with that for the high portion because each may be affected by the other. Joints should be provided to prevent cracking, but their locations are influenced by the nature of the system itself.

Sometimes a designer is faced with a situation where the lateral forces are so great that inevitable movement should be provided for without endangering the main structure. Figure 8-41 shows, in simplified form, part of the layout of a large smelter where this was done. The drawings illustrate the basic features of the design. The reverberatory furnaces are over 100 ft. long. When such a furnace is in operation, the heat causes expansion, and the force will be tremendous if motion is not permitted. Furthermore, when a furnace is cooled down, it will seldom return to its original length but will crack. Rebuilding and reheating a furnace are likely to cause longitudinal creep of the foundation. Therefore, the main steel superstructure is supported upon towers that have a heavy combined foundation, a sort of island, as indicated in Fig. 8-41(a). In front of each furnace is a light retaining wall, as shown in principle in (*c*). This wall may slide or tip as a unit without endangering anything except itself, and it may be rebuilt if necessary.

8-14. Anchorages for pipe lines. Pipe lines that contain rapidly flowing liquids, or even static fluids subjected to large pressures, need special foundations. The vertical dead loads may be moderate; the thrusts caused by pressure and flow may be relatively important.

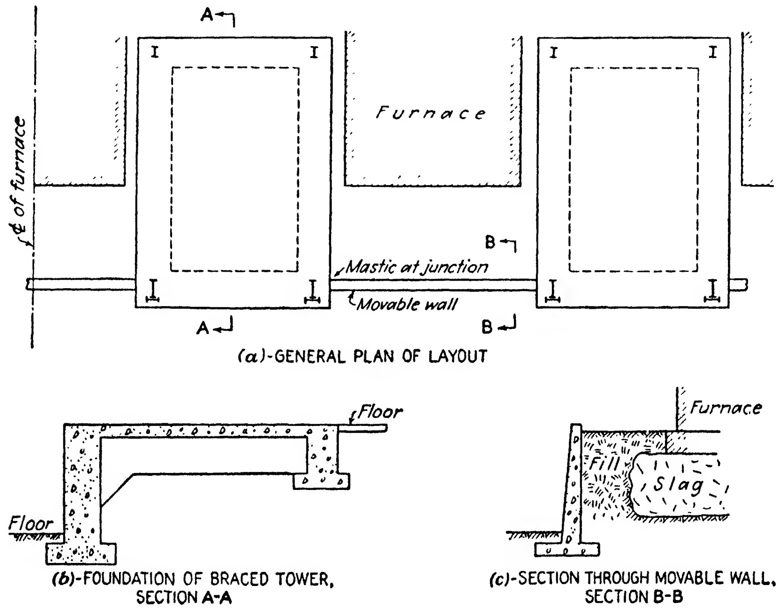


FIG. 8-41. A situation in which the main structure is isolated from the part that is subjected to serious lateral forces.

As a starting point for a study of these problems assume the portion of a pipe line shown in Figs. 8-42(*a*) and (*b*). The pipe is 24 in. O.D., $\frac{3}{8}$ in. thick, welded steel, and the maximum flow expected is $Q = 80$ c.f.s. The pressure head h at B when the water is shut off at the discharge end is 190 ft.; when the water is flowing, it is 175 ft.

First, consider the vertical load of the pipe and its contents. Of course the span AB should not exceed that which the pipe can stand safely as a beam to hold up the weight of the water and the steel. A small pier or pad would be suitable for supporting the vertical load of a straight pipe at such a point as A, since $P = (95 + 2.95 \times 62.5)50 = 14{,}000$ lb. (approx). Nevertheless, the pipe should not rest upon a flat concrete surface as shown in Fig. 8-42(*d*) because of the local concentration of pressure. When a cradle is built of concrete, as in (*e*), there should be some protective or cushioning material between the pipe and the masonry, and it is generally desirable to anchor the pipe more or less securely by some such means as the anchor-bolted strap shown in (*e*).

Next, consider the effect of changes in temperature. The pipe is made ⅜ in. thick in order to have strong beam action and a reserve allowance against corrosion, not just because of pressure. Assume that the range of temperature for span AB is only $\pm$ 25°F. from normal. Then the total change in length between A and B due to expansion and contraction is

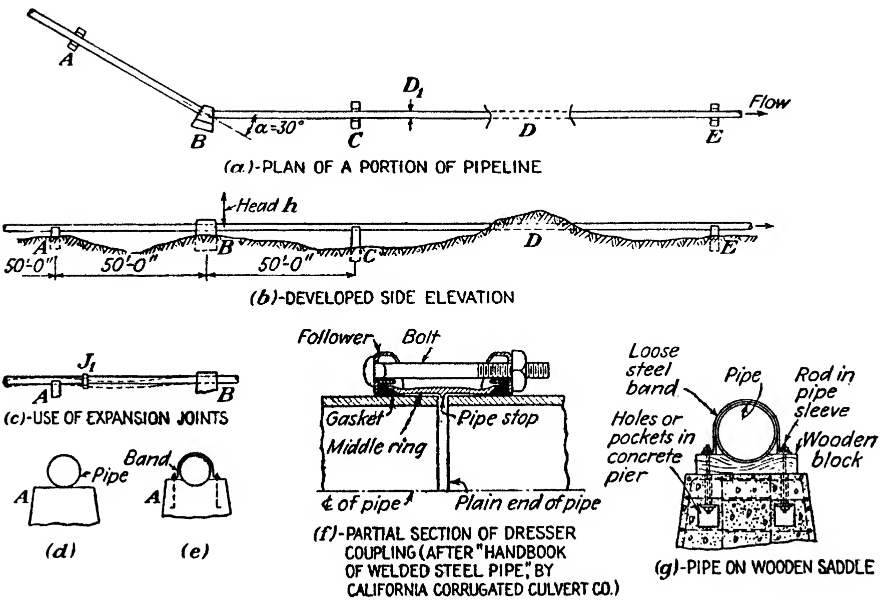


FIG. 8-42. Pipe lines and foundations for them.

approximately $\Delta L = 0.0000065 \times 50 \times 12 \times 50 = 0.195$ in. If the pipe is fastened firmly at B, it will try to slide on top of the foundation at A, or to tip this foundation slightly. One can see that, if a continuous pipe extends over a series of spans, this movement near the end may become considerable and troublesome. On the other hand, if the pipe is supposedly anchored immovably at each support, the theoretical unit stress in the pipe for a change in temperature of 25°F. either side of the average is

$$f_s = E\delta = 30{,}000{,}000 \times 0.0000065 \times 25 = \pm\ 4{,}900 \text{ p.s.i.}$$

The maximum total force that the entire restrained pipe might exert upon its foundations is, therefore,

$$F = \pm\ 4.9 \times \pi \times 23.62 \times 0.375 = 136 \text{ kips}$$

An examination of the preceding paragraph shows that provisions should be made to permit longitudinal motions and thereby eliminate these large temperature stresses and forces. One way to do this is to install some kind of slip joint in the pipe line, as indicated by J in Fig. 8-42(c). One type of joint for such use is shown in (f). However, as indicated to exaggerated scale by the dotted lines in (c), these joints may interfere seriously with the beam action of the pipe unless the joints and the foundations are located properly with this in mind.

One possible arrangement for a straight pipe line is the use of an anchorage near the center of a long length of pipe, with rocker bents each side to resist vertical and transverse forces but to permit longitudinal motion to be taken up at the expansion joints. Another is the use of the anchor pier with adjacent foundations on which the pipe may slide. Unless the pipe is on rollers or can slide upon a wooden or other saddle as shown in Sketch (g), the pipe is likely to be worn harmfully. If the pipe is embedded in soil, as shown at D in Fig. 8-42(b), this may be a suitable anchorage in some respects. The frictional resistance of the soil on the pipe will hold it unless the friction is overcome.

Next, consider what happens at an elbow or curve, such as at B in the pipe line of Fig. 8-42(a) because of pressure. Assume that the pipe is welded as shown in Fig. 8-43(a). It is clear that the hydrostatic pressures on the inside of the pipe will counteract and be in equilibrium on the left side of the section GJ and on the right of HK. The pressure on the junction

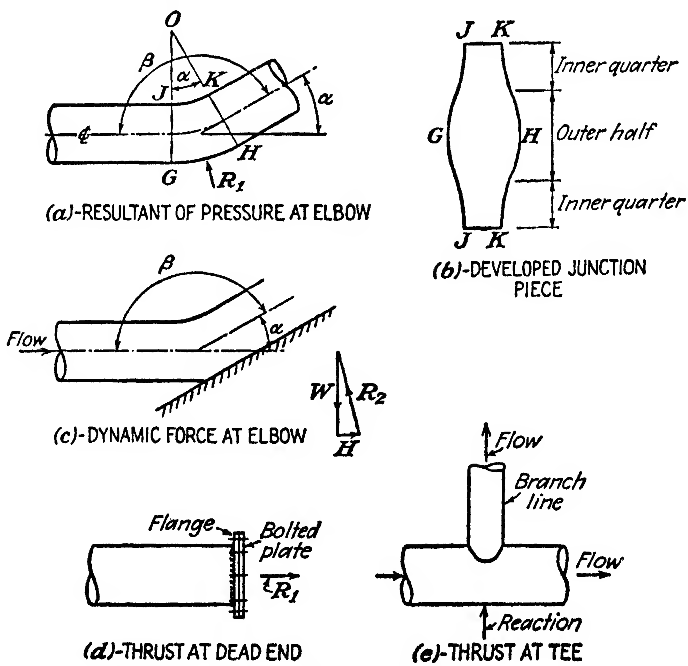


FIG. 8-43. Forces at elbows and bends in pipe line.

piece, however, is not balanced. This piece is shown unrolled in (*b*). A resultant force R_1 is therefore present at the elbow. Then,

$$R_1 = 2pA(\sin \tfrac{1}{2}\alpha) \tag{8-23}$$

where p = hydrostatic pressure in pounds per square foot or $62.5h$, A = the cross-sectional area of the inside of the pipe in square feet, and α = the deflection angle of the elbow. Therefore, for the maximum head of 190 ft.,

$$R_1 = 2 \times 62.5 \times 190 \times 2.95(\sin 15°) = 18{,}150 \text{ lb.}$$

For a head of 175 ft., $R_1' = 16{,}700$ lb. This resultant force acts in the plane of the center lines of the two pipes, and along the bisector of the angle β, Sketch (*a*). The components of the pressure normal to the plane of the elbow counteract each other. When the bend has a long radius, one should not forget that the lateral pressure is spread over the region represented by the space between *GJ* and *HK* of Fig. 8-43(*a*). Therefore, the anchorage should embrace this area.

When an elbow or bend is encountered by flowing water in a pipe line, the inertia force of the moving fluid causes a force that tends to push the bent portion of the line outward, as represented by R_2 in Fig. 8-43(*c*). If Q = the quantity of water flowing past the elbow in cubic feet per second, w = unit weight of the fluid in pounds per cubic foot, v = the velocity of flow in feet per second, and g = the acceleration caused by gravity,

$$R_2 = \frac{mv \sin \alpha}{\cos \frac{1}{2}\alpha} = \left(\frac{Qw}{g}\right) v \frac{\sin \alpha}{\cos \frac{1}{2}\alpha} \quad \text{or} \quad 2\left(\frac{Qw}{g}\right) v \sin \tfrac{1}{2}\alpha \tag{8-24}$$

This force is coincident with R_1 of Sketch (*a*), neglecting eddies and frictional losses. Then, for point *B* of Fig. 8-42(*a*) and $\alpha = 30°$,

$$R_2 = \frac{80 \times 62.5}{32.2} \times \frac{80}{2.95} \times \frac{0.5}{0.965} = 2{,}180 \text{ lb.}$$

Therefore, the greatest thrust at *B* occurs when the water is flowing, since $R_1' + R_2$ exceeds R_1.

Of course, a dead end like that in Fig. 8-43(*d*) will have a longitudinal thrust $R_1 = pA$ because the flow is necessarily zero. A tee like that in Sketch (*e*) must be anchored to resist both the pressure and the dynamic force as determined by Eqs. (8-23) and (8-24), using the cross-sectional area of the branch line. In the case of a diagonal branch and other special situations, the magnitude and direction of the thrust should be determined for the particular angles, sizes, pressure, and flows that apply to each specific problem.

If the pipe line has expansion joints in it to relieve the anchors of the thrusts produced by changes in temperature, the principal forces to be resisted by the anchorages are those represented by Eqs. (8-23) and (8-24).

The frictional resistance of the pipe to the flowing liquid, water hammer, the hydrostatic pressure on the exposed end of the pipe at the expansion joint, and the frictional resistance of the pipe if it is forced to slide over adjacent piers—these are forces that may be of importance in extreme cases. However, a conservative safety factor of 2 in the design of the anchorages generally covers all these minor forces with a satisfactory margin.

When a pipe line is buried in a relatively deep trench, the weight of the soil backfill is generally sufficient to hold a straight pipe line in place without frequent expansion joints. At an elbow or sharp bend, a pad of concrete should be placed as shown in Fig. 8-44(*a*). When a horizontal bend, as shown in (*b*), is made by a series of pipes connected by expansion joints, and each pipe is skewed slightly at the junction within the limits permitted by the type of fitting used, compacted backfill along the outside of the pipes is generally sufficient to resist the thrusts.

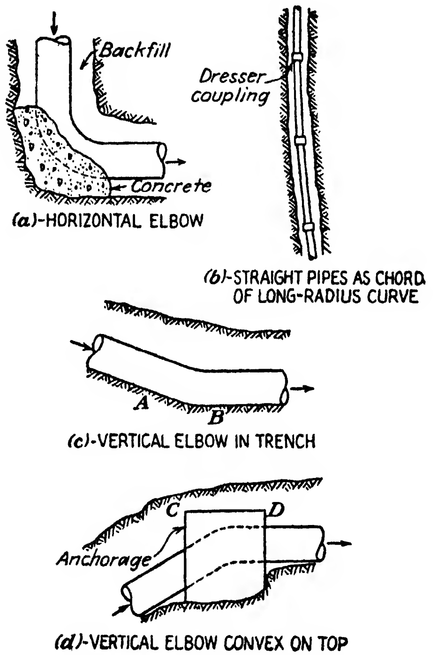


FIG. 8-44. Pipes set in trenches.

An elbow in a vertical plane, if the bend is concave up as shown in Fig. 8-44(*c*), is generally safe automatically because the thrust is downward. However, it is important that the soil bear firmly against the underside of the pipe at the bend. If tamped soil is not considered trustworthy or if points of rock may cause localized punching, a concrete pad or cradle under the pipe in the vicinity AB is desirable. If the soil is muck or other unreliable material, a wide concrete anchor, a group of piles with a concrete pad, a stone-filled timber crib, or some other device should be used to spread the thrust and support the structure safely.

An elbow in a vertical plane with the bend convex upward may be difficult to restrain. Figure 8-44(*d*) illustrates such a case. The earth over the region CD may not be sufficient to resist the upward thrust unless the trench is very deep or a large mound of earth is built over the bend. Even then, the weight of the soil should not be trusted because, when some maintenance man thoughtlessly removes the soil, the pipe line may be damaged. It is advisable to anchor the line safely as though it were exposed.

The schematic drawings in Fig. 8-45 are intended to provide a few suggestions that may help the reader in designing pipe supports and anchorages.

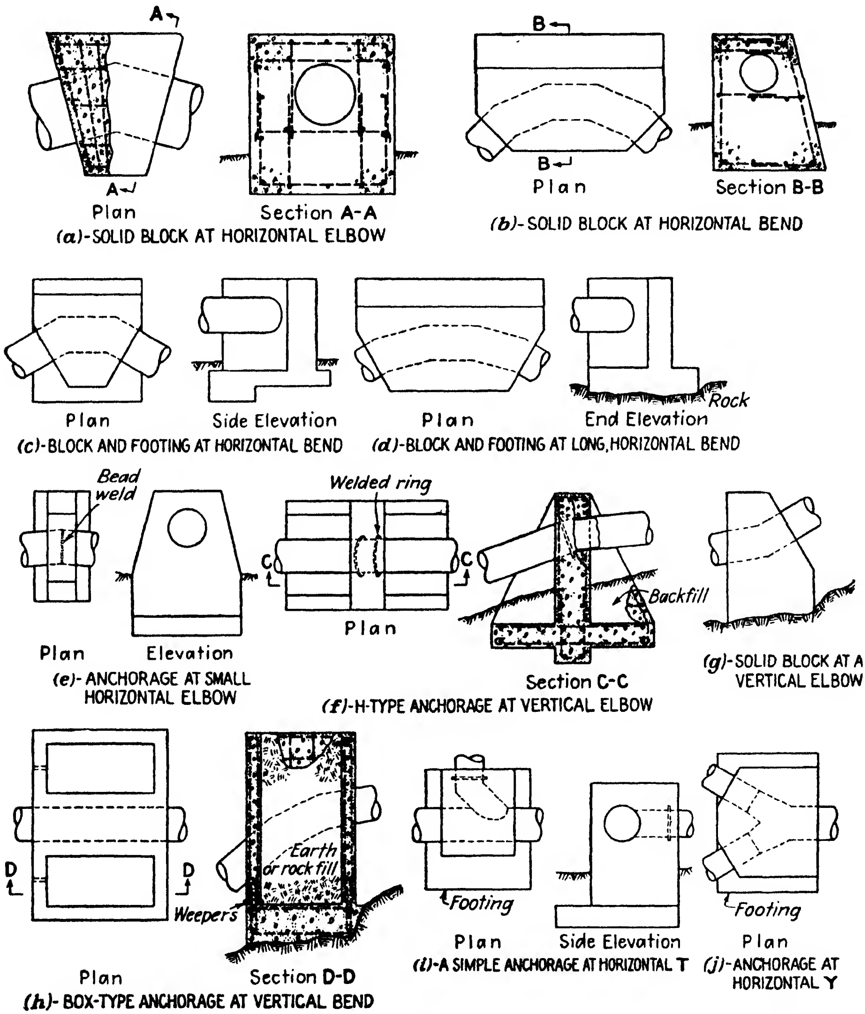


FIG. 8-45. Some types of anchorage for welded steel pipe lines on earth or rock. (*Courtesy of A. L. Otto, Anaconda Copper Mining Co.*)

When a pipe line is built over rough terrain, he will probably encounter a great variety of conditions. The bends are likely to be in inclined planes, and he must design the supports and anchorages to fit the local requirements. Pipe trestles and bridges may be needed; tunnels in earth or rock may be worth their cost; even a catenary action of the pipe itself with special anchors might be used across a deep ravine. Control valves, branch lines, siphons, points for access and egress of go-devils or other cleaning devices, removal of sections for repairs or maintenance, by-passes, manholes, loads from overpassing traffic, floods, erosion, frost, and corrosion from without and within these are among the factors that the designer of a pipe line should consider.

PROBLEMS

8-1. Assume the column shown in Fig. 8-46. Are the anchor bolts satisfactory if $P = 290$ kips, $e = 6$ in., and $M = 220$ ft.-kips? Assume that the allowable tension in the anchor bolts is 12 k.s.i. at the bottom and 18 k.s.i. at the threads. Assume the point of rotation to be under the back of the 12-in. channel.

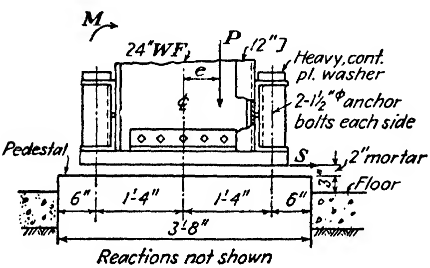
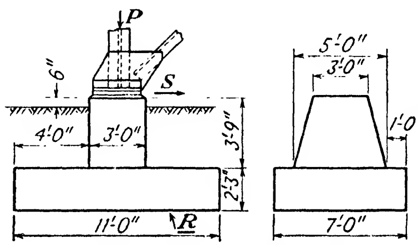


FIG. 8-46. Base of an interior crane column in a steel mill building.



(a)-SIDE ELEVATION (b)-END ELEVATION

FIG. 8-47. Footing for a viaduct column.

8-2. Check the footing shown in Fig. 8-47 for safety under the following conditions:

$P = 90$ kips min; 300 kips max
$S = 0$ kips min; 42 kips max from wind
Soil = sandy clay having allowable $p = 5$ k.s.f.
Safety factor = 2
Max coefficient of friction $f = 0.75$

Suggestions: Test for overturning, sliding, and edge pressure for the following cases:

1. P_{min} and $2 \times S_{max}$, with limiting $p = 2 \times 5$ k.s.f.
2. P_{max} and S_{max}, with limiting $p = 1.3 \times 5$ k.s.f.
3. Finally, test for P_{min}, $2(P_{max} - P_{min})$, and $2 \times S_{max}$, with limiting $p = 2 \times 5$ k.s.f.

Case 1 is the only one in which sliding may be critical.

An easy way to do this problem with sufficient accuracy is to compute the weight W of the footing and the soil on it, then divide W, P_{min}, P_{max}, S_{max}, $2(P_{max} - P_{min})$, and $2 \times S_{max}$ by 7, the width of the footing normal to S. Use these quotients as loads on a strip of footing 11 ft. long and 1 ft. wide. On a drawing of this strip of footing draw the loads to scale for various combinations, and scale off eccentricities, etc., as shown in Fig. 8-48. Compute the ordinates of the respective pressure diagrams.

Ans. Edge pressure for case 3 is 10.9 k.s.f., a bit too high. Otherwise the footing is safe.

8-3. Design an isolated spread footing for the steel viaduct column shown in Fig. 8-49, assuming the following data:

$$P_{DL} = 140 \text{ kips}; P_{LL} = 180 \text{ kips}$$

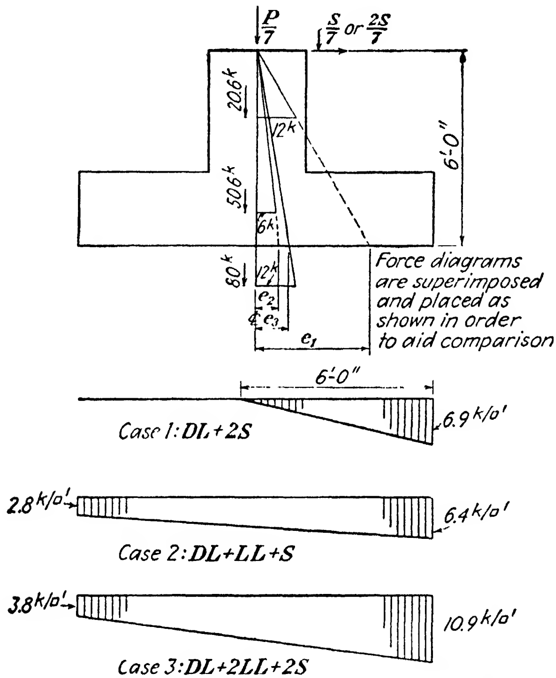


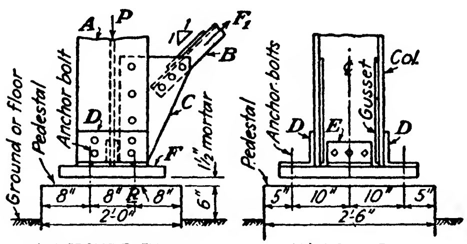
FIG. 8-48. Analysis of footing, using graphics to determine eccentricity.

Tension in bracing F_1 max = 50 kips for longitudinal braking forces and wind combined as a live load.

Soil = medium sandy clay having allowable p = 6 k.s.f.
Safety factor = 1.5
Location—Cleveland, Ohio

Suggestions: The elevation and size of the top of the pedestal are shown. The bottom of the footing must be below frost depth. The force F_1 is the load in member B. Its horizontal component is the force to be used as S, and S is assumed to be transmitted through the gusset C to a working point at the intersection of the center lines of A and B. Assume that this working point is 8½ in. above the bottom of the billet F. Then S acts 10 in. above the pedestal. The vertical component of F_1 counteracts some of P. Make a guess at the weight W of the footing, add it to $P_{DL} + P_{LL}$, divide the sum by p to compute an area, and use a trial area some percentage larger than this (perhaps 20 per cent). Assume dimensions for the footing and recompute W. Test for $P_{DL} + P_{LL}$ and F_1 max, with a limiting $p = 6$ k.s.f., since F_1 is not entirely a wind load but a force that exists with wind and live load P_{max}, or with wind combined with live loads in adjoining spans and affecting this braced bay. Finally, test for P_{DL}, $1.5P_{LL}$, and $1.5F_1$ max; then for P_{min} and $1.5F_1$ max.

8-4. Assume the construction shown in Fig. 8-50. Neglecting the lateral supporting value of the floor and earth, assuming the pedestal to be 3 ft. high, and



(a)-FRONT ELEVATION (b)-SIDE VIEW

FIG. 8-49. Base of a braced column.

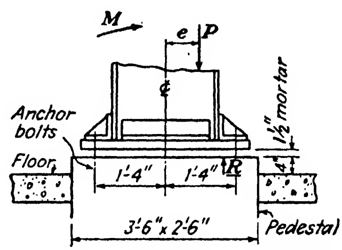


FIG. 8-50. Base of an interior crane column in an industrial plant.

assuming a footing 9 ft. square and 2 ft. thick centrally located, compute the maximum pressure on the soil for the following case:

$$P = 375 \text{ kips} \qquad e = 6 \text{ in.} \qquad M = 50 \text{ ft.-kips}$$

Ans. 7.2 k.s.f.

8-5. Assume the construction shown in Fig. 8-46. Neglecting the lateral supporting value of the floor and earth, design an isolated spread footing for this column for the following conditions:

$$P = 320 \text{ kips} \qquad e = 7 \text{ in.} \qquad M = 90 \text{ ft.-kips} \qquad S = 5 \text{ kips}$$
$$\text{Allowable } p = 7 \text{ k.s.f.} \qquad \text{Safety factor} = 1.5$$

8-6. Design an isolated spread footing for the combined interior steel columns shown in Fig. 8-51. Column A, a building column in an industrial plant, supports a bracketed crane girder on its left flange; B supports a crane girder that is seated upon its top over and parallel to the web. The diaphragms C will not transmit vertical loads from one column to the other. Assume the following design data:

$$P_1 = 60 \text{ kips DL; } 110 \text{ kips LL}$$
$$P_2 = 10 \text{ kips DL; } 250 \text{ kips LL}$$
$$M = \pm 100 \text{ ft.-kips LL; } 0 \text{ ft.-kips W}$$
$$S = \pm 10 \text{ kips LL; } 0 \text{ kips W}$$

The bulk of all lateral forces is resisted by roof bracing and vertical bracing located elsewhere. Assume that the diagonals of the longitudinal bracing towers for the columns are not connected to these particular columns. The soil has a safe bearing value of 8 k.s.f. The safety factor is 1.5.

Suggestions: In this case, the live load can be on either column alone, or on both. Design the footing for the maximum loading conditions, then analyze it for the maximum live load upon B only. Assume that 30 per cent of M and S is caused by live loads on A; 70 per cent on B.

8-7. Figure 8-52 pictures a double steel column to be used as the support for two adjacent crane runways in the interior of a large industrial plant. The roof is supported by a stub column connected to a pair of heavy channels across the tops of the crane columns. Either crane may be loaded at a time, and both may be loaded simultaneously. Assume the following design data:

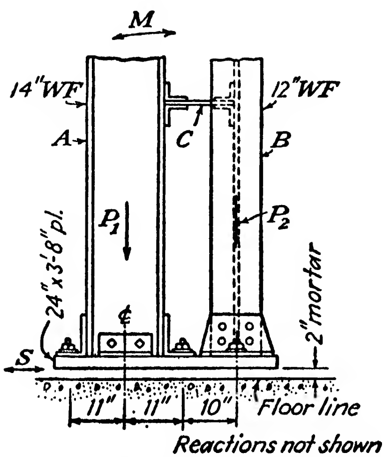


FIG. 8-51. A double column for two adjacent crane runways.

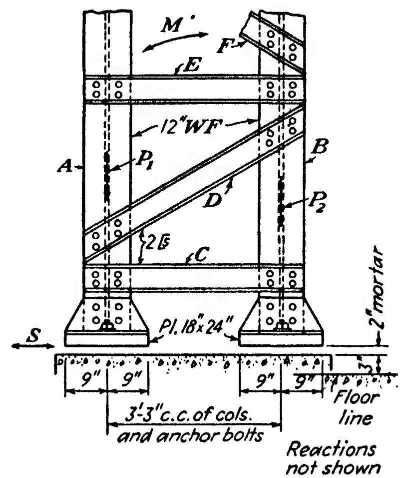


FIG. 8-52. A heavy double column for two adjacent crane runways.

$$P_1 = 35 \text{ kips DL; } 170 \text{ kips LL}$$
$$P_2 = 35 \text{ kips DL; } 170 \text{ kips LL}$$
$$M = \pm 900 \text{ ft.-kips LL; } \pm 150 \text{ ft.-kips W (W = wind)}$$
$$S = \pm 30 \text{ kips LL; } \pm 5 \text{ kips W}$$
$$\text{Safety factor} = 1.5$$

The soil is a sandy clay that will support safely a unit pressure of 6 k.s.f. The columns are laced together so strongly that the participation stresses in the lacing are capable of practically equalizing the live load in the columns for varying load conditions on the cranes. Furthermore, the cantilever action of the braced columns is relied upon to withstand lateral live load and wind forces. The columns are braced strongly in the direction of the crane girders by towers in special bays, not at the point shown. Design a combined isolated footing to support these columns.

Suggestions: Determine the maximum load combination, and design the footing to support the vertical and overturning forces without reliance upon lateral support from the soil or the floor slab. Notice that the wind forces may occur with dead load only and with live loads as well.

8-8. Figure 8-53 shows a tall steel bent that is to be used to support two long trussed spans of a heavy twin-conveyor gallery at an industrial plant. Two railroad tracks pass diagonally under its center. Assuming the steelwork to be designed properly, check the pressure and uplift on the proposed footings if the loads and conditions are the following:

P_1 = truss reactions = 100 kips DL and 100 kips LL
P_2 = weight of bent = 16 kips DL
S = wind load = 30 kips
W = weight of footing and earth = 25 kips assumed at A and B
p = 8 k.s.f. allowable
Safety factor = 2

Suggestions: The legs or columns of the bent are inclined, and the direct loads in them will be assumed as parallel to their longitudinal axes. Assume that the shear S is resisted equally by A and B as a horizontal force applied at their tops, and that each footing is set normal to the axis of its column so that the horizontal

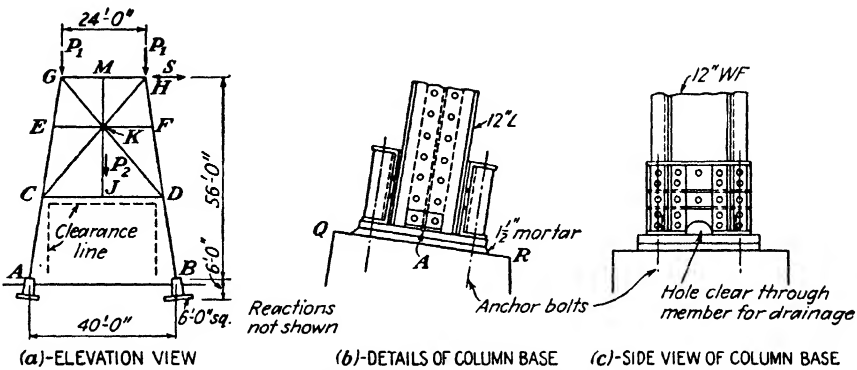


FIG. 8-53. A tall bent supporting long, twin conveyors over railroad tracks.

component of the column reaction will not tend to tip the foundation. The sloped bottom of a column avoids the use of bent details at the base of the steel. The anchor bolts are to be parallel to the column shaft whether the top of the pedestal is sloped or the inclination is secured by using a wedge-shaped pad of mortar or grout under the steel.

8-9. A conveyor trestle is to be built across part of an old dried-up tailings pond. A proposed design for the foundation of a typical single bent is shown in Fig. 8-54. The original ground is fine sand, which can withstand a pressure of at least 6 k.s.f. The tailings material can be excavated easily and will stand on practically vertical slopes temporarily, whereas its supporting value is very small and untrustworthy. No piles and pile-driving equipment are available. Is the design for the substructure satisfactory for a safety factor of 1.5? The lateral supporting power of the tailings around the concrete may be utilized.

Suggestions: Compute the maximum pressure under a footing, as though located directly at A or B. The lateral supporting power of the backfilled tailings will hold the substructure as though it were an integral part of the earth, and the pier cannot tip about its bottom. For resistance to uplift, one-half of the pier and the weight of the soils above a footing like EF may be relied upon. Since the sand is confined and its bearing value is probably increased by the weight of the overburden, the footing appears to be adequate for pressure requirements. It is needed for securing the necessary resistance to uplift, and the footing should be increased because the safety factor of 1.5 is not obtained. In such soft material, the top strut is probably worth its cost.

8-10. A steel tower or headframe with two bents and transverse trusses supports two large sheaves around which run the cables for a two-car inclined skipway from the underground workings of a large mine. Figure 8-55(a) shows the general framing, dimensions, and forces to be used for design purposes. The rock is sloping as shown, but the area around the structure will be filled in to form a level working area. An engineer has designed the foundations shown in (c), (d), and (e) for the

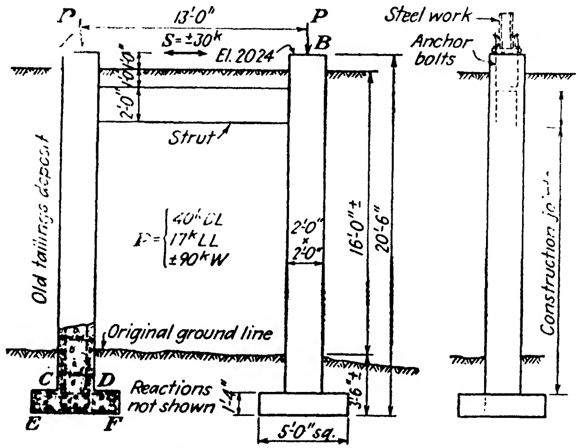


FIG. 8-54. Tall pier embedded in deep fill.

structure. The forces shown are for one bent of the tower. If the safety factor is at least 1.5, check these foundations to see if they are satisfactory.

Suggestions: The bearings at A and B are planned to resist vertical forces and some transverse wind shears, whereas that at C is to transmit both vertical and horizontal forces into the rock. The framework is assumed to be adequate in strength. The forces applied to the foundations are indicated in Sketch (b). The tower will tend to overturn about C, a rigid support. Multiply the resultant R of the cable pulls and the wind load by 1.5. Take moments about C, and include the righting moment of the dead load W. Compute the moment of inertia of the bearings about C as follows: $I = 1 \times 32^2 + 1 \times 50^2$. The uplift at A is $M \times 50/I$; at B, $M \times 32/I$. This assumes that the bearings and anchor bolts at A and B are substantially equal, and that the tensile unit stresses in them are proportional to their distances from C. Notice that this assumption is different from what it would be for a section cut through an elastic framework or medium.

Ans. $U_A = 50$ kips; $U_B = 32$ kips; $D_C = 136$ kips; $H = 122$ kips. The foundations are satisfactory.

8-11. A notched footing similar to that in Fig. 8-18 has the following dimensions and loads: footing, 9 ft. square; notch in one corner, 1 ft. wide and 2 ft. 6 in. long;

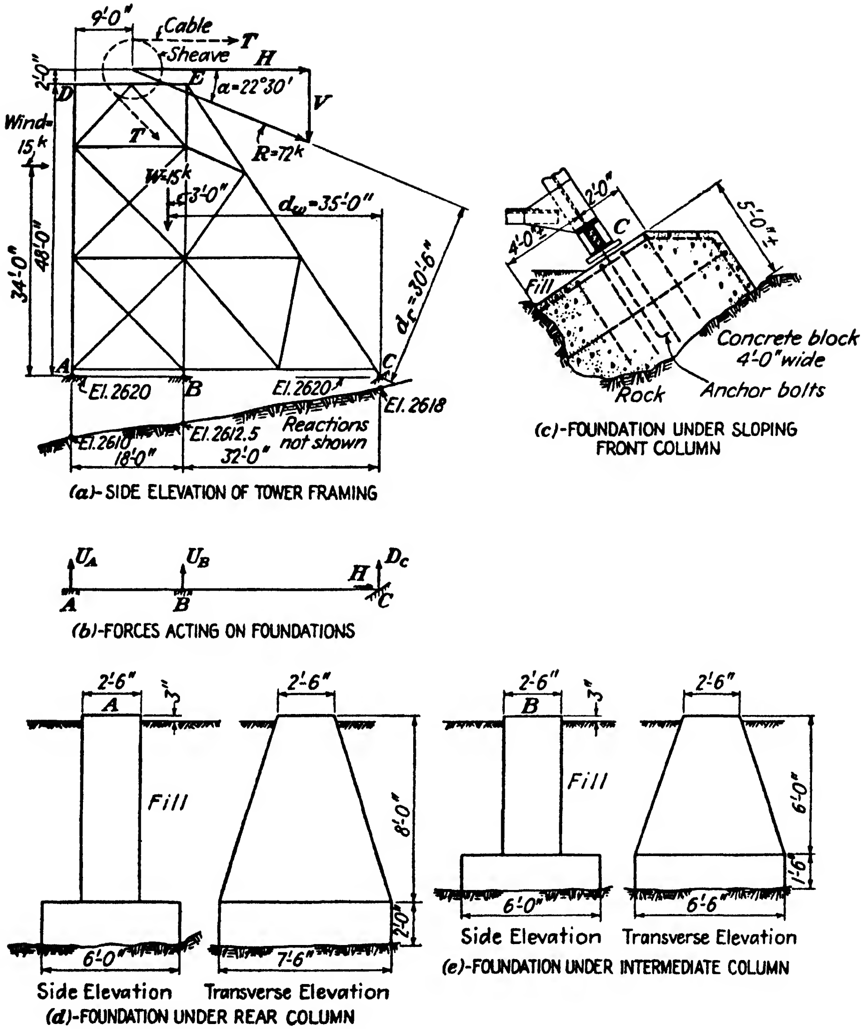


FIG. 8-55. Foundations for a headframe at an inclined mine shaft.

weight of footing and column combined, 440 kips, assumed applied at the center of the pedestal. Compute the ordinates of the pressure diagram.

8-12. A stepped footing is 10 ft. square and 2 ft. thick. The pedestal is 3 ft. square, 4 ft. high, and centrally located, with its top 1 ft. above the ground. It supports a concrete column having a maximum load of 500 kips, not including the footing. The soil is good for a bearing pressure of 6 k.s.f. The owner wishes to cut a strip 12 in. wide and 18 in. deep off one entire side of the spread footing so that he may install a conduit trench. Should you approve such action if the safety factor for the soil is 1.5?

Suggestions: The 6-in. concrete layer theoretically remaining under the cut will be so badly damaged that it should be considered worthless after the other concrete is knocked off. Find the maximum pressure as though the footing were 10 × 9 ft., with the load 5 ft. from one side and 4 ft. from the other. Consider the revised weight of the footing and its eccentricity.

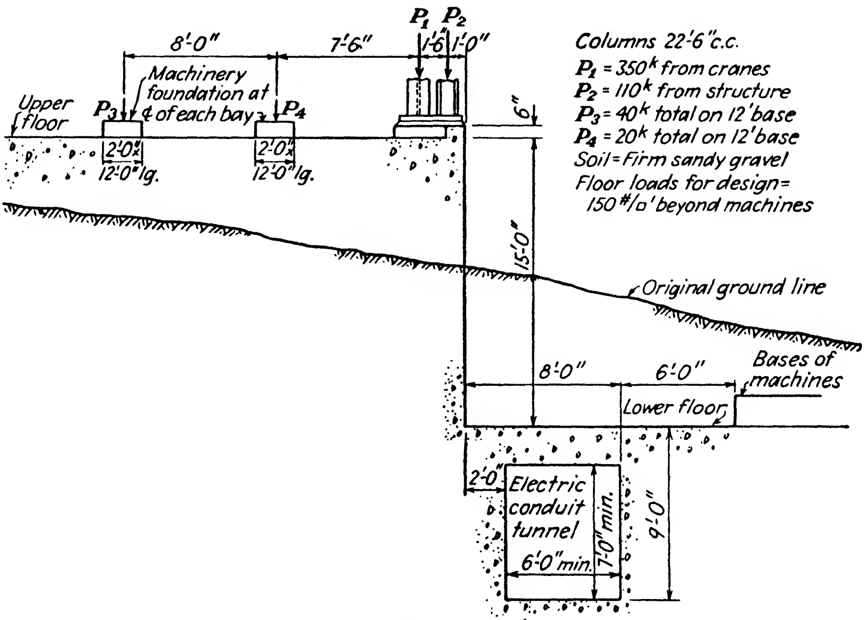


FIG. 8-56. Problem in planning a retaining wall to support offset floor in an industrial plant.

Ans. $p_1 = 8.1$ k.s.f. The footing may not fail but the proposed change should not be made. It will reduce the safety factor to $(6 \times 1.5)/8.1 = 1.11$.

8-13. Design a gravity anchorage at a 30° vertical bend in a pipe line similar to that shown in Fig. 8-45(f). The static head is 180 ft. when the discharge is 200

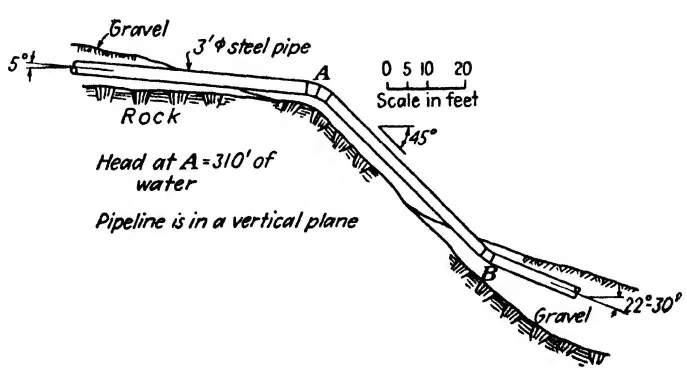


FIG. 8-57. Problem in planning anchorages for a pipe line.

c.f.s. The pipe is 30 in. O.D. and $\frac{3}{8}$-in.-thick welded steel, and the elbow is made by welding sections so that the inner radius approximates 5 ft. The safety factor is to be 2. The soil is a layer of gravel 3 ft. thick overlying rock. The location is at the edge of a ravine in New Mexico. The water is running down the 30° slope of the ravine. The expansion joints on each side of the anchorage are 100 ft. away, and the pipe is supported upon several intermediate piers having a small coefficient of friction. The anchorage is to be designed for pressure and dynamic forces only.

8-14. Plan the construction to support the columns and machinery bases shown in Fig. 8-56 and to form the 15-ft. wall. The tunnel is to be of reinforced concrete. After developing a plan, design a typical portion of the structure. The electric tunnel runs the full length in front of and below the wall.

8-15. Assume that the pipe line shown in Fig. 8-57 is in a vertical plane. Design the anchorages at A and B for the head shown, using the static pressure only.

8-16. Compute the edge pressure at AB of the footing of Fig. 8-17A if P is 250 kips and S is 6 kips. Include the weight of the earth and the substructure.

9

PILES

9-1. Function of piles. This chapter deals with piles primarily as a material or element in a foundation. Basically, they are part of the substructure. They may be used to transmit forces from the superstructure and the remainder of the substructure through weak unsuitable soils, water, or air to a lower stratum that has sufficient bearing value to support the completed structure and all the loads that are expected to be applied to it; or they may be used to spread these loads through and over sufficient relatively weak soil to enable it to support the structure safely.

Obviously, piles are used when the subsurface conditions are not suitable for the use of spread footings and mats, or when it is not economical and preferable to build these last in the positions where they should be located if the piles were not used. Piles are therefore generally associated with difficult foundation problems and with dangerous subsurface conditions. Nevertheless, this does not mean that pile foundations are hazardous. This is a warning to inexperienced and unwary persons, particularly the owner and the person responsible for the design. Planning a pile foundation—and frequently the construction of it—require that one obtain all the data about conditions at the site that can be secured reasonably, that he investigate and compare all feasible plans for the foundation, that he eliminate all uncertainties that can be avoided, and that he respect and use the sound engineering judgment that is developed through study and experience.

"Piles are good as foundations. Therefore, these piles are good." This is logical. However, piles are no better than the materials to which they transmit their loads. One cannot accomplish much by pushing on a rope, neither should he expect a pile to offer adequate resistance when it has nothing to produce the necessary reaction. In pile-foundation problems, therefore, the soils under and around the piles are the things that need to be studied carefully. Obviously, the piles themselves must be long enough, strong enough, durable, and suitable for the particular conditions in which and for the structure for which they are to be used. Furthermore, the intended construction should be practicable.

What is meant by failure of a pile? Not necessarily does a pile have to vanish into subterranean depths, nor does it have to break and fold up like a jackknife to be a failure. When a pile cannot support the intended or needed loads without a subsidence that renders the structure unsafe, unusable, or so damaged as to impair its value greatly, then it may be said to have failed. Furthermore, a settlement that constitutes practical failure in one case may not be vitally harmful in another. Some settlement of a pile foundation not carried to rock is almost inevitable; if excessive expense is entailed, the settlement may be kept very small. However, unnecessary cost and uneven subsidence are to be avoided. What the particular circumstances are makes a tremendous difference. An accurate predetermination of the settlement is difficult to make. Unfortunately, one cannot try out a pile foundation, then change it at will if it proves unsatisfactory. If it is safe and serviceable, the owner will be pleased; if not, he will forget that he may have applied pressure to have the costs reduced.

In general, service proves whether or not a pile foundation is safe. Whether a given foundation could have been made safely at less cost is a subject for argument without much chance for proof. Many and frequently difficult decisions have to be made in the planning of pile foundations.

9-2. General action of a pile under load. A pile may be driven into a deep stratum of granular or cohesive soil, as pictured in Fig. 9-1(a). When the load P is applied to the top or butt end, the pile tends to penetrate farther into the soil. Compression and displacement of the material under the tip B offer some resistance, but most of the reaction is caused by tangential *skin friction* along the surface of the pile, as represented by ΔF. Hence it may be called a *friction pile.*

The magnitude of the coefficient of friction for the contact surface of the soil and pile depends mostly upon the properties of the soil. The character and smoothness of the surface of the pile may have some effect, but they seem to be relatively unimportant except where the bark is left on wooden piles, in which case the bark may loosen and isolate the pile from the soil. Naturally, the frictional resistance per unit of area of the pile will depend upon the intensity of the normal pressure, which is likely to vary in some relation to the depth to the point considered. It is upon this basis that Table 9-1 was prepared. It is intended as a qualitative guide, not as correct information to be used in any and all cases. Actual values depend upon the properties of a particular soil, and its resistance to shear and compression should be studied by means of laboratory tests on undisturbed samples. The combined effects of friction and cohesion are classed herein as frictional resistance.

The total load P on the pile in Fig. 9-1(a) must be resisted by $\Sigma\Delta F$ if this can be done. The total resistance will then be

$$R = -P = \Sigma(\Delta A)(pf + c) \tag{9-1}$$

where p and f are the unit normal pressure and the coefficient of friction, respectively, acting upon a specific unit of area and c is the unit cohesion. The area of the surface of the pile per foot of length may or may not vary, depending upon its dimensions; when tapered as in Sketch (a), the

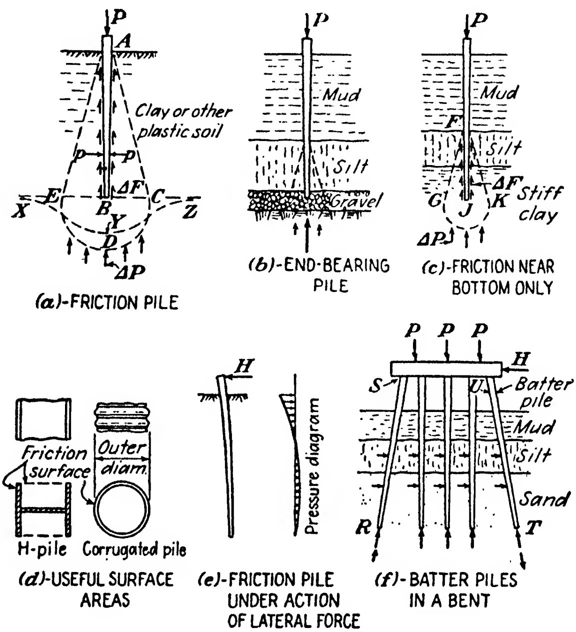


FIG. 9-1. Approximate action of piles when resisting loads.

taper itself may have some, but a rather minor, beneficial effect upon the friction, but the smaller tip and gradually changing cross section may have great influence upon the ease of driving the pile. This taper, however, may cause the resisting normal pressures on the surface of the pile to have an appreciable vertical component that is helpful, although the taper is usually too small for this action to be dominant.

Not only is the frictional resistance at the surface of the pile important but so is the shearing resistance of the surrounding soil against sliding upon itself. Imagine that the pile in Fig. 9-1(a) settles slightly into the ground. Even though the soil may not slide along the actual surface of the pile (the corrugated shell of a Raymond concrete pile, for instance), it may slide upon itself along a curved area just outside the pile's surface. Successive concentric areas located outward from the pile must also transmit the forces

TABLE 9-1. Approximate Allowable Value of Skin Friction on Piles*

Material	Skin friction, p.s.f.		
	Approx depth 20 ft.	Approx depth 60 ft.	Approx depth 100 ft.
Soft silt and dense muck........	50–100	50–120	60–150
Silt (wet but confined)..........	100–200	125–250	150–300
Soft clay.....................	200–300	250–350	300–400
Stiff clay.....................	300–500	350–550	400–600
Clay and sand mixed..........	300–500	400–600	500–700
Fine sand (wet but confined).....	300–400	350–500	400–600
Medium sand and small gravel...	500–700	600–800	600–800

* Some allowance is made for the effect of using piles in small groups.

from one assumed thin film to the next, but the areas of these surfaces of possible failure increase so that the critical surface is likely to be that one located just outside the pile. As a result of the transfer of loads from one curved layer to the next curved layer by shear, and from horizontal stratum to horizontal stratum by compression, one may assume that, for convenient visualization on his own part, the pile in Sketch (*a*) distributes its load through some volume of material, such as that shown by section $AEDCA$. This will be called the *cone of distribution* of the load. In other words, the concentrated load P is spread by the use of the pile to an area of soil below the circle whose diameter is EC. The angle BAC is not known exactly, and AE and AC may not be straight lines, but this concept of the spreading of the load is useful in planning and analyzing pile foundations.

Probably the unit pressure is greatest near B of Sketch (*a*), hence assume that the circular arc EDC represents the pressure diagram for the soil at the general level of B. Call this imaginary hemispherical bottom the *bulb of pressure* produced by the pile and its load. Such a picture as that of Fig. 9-1(*a*) is not intended to imply that the pile has no effect upon the soil below EDC. However, as the pressures are distributed outward and downward, the intensity of pressure upon the soil decreases to a value too small to cause appreciable settlement. It may be that area XYZ is a truer representation of the character of the pressure diagram on the soil at the level of the point of the pile. However, the illustrations are to give the reader qualitative rather than quantitative data.

These two concepts may be helpful when one tries to get a qualitative mental picture of the action of piles. Correct quantitative data may be difficult to obtain.

Sometimes a pile is primarily a column that supports a load on its top and in turn bears upon a firm stratum at and near its tip, as illustrated in Fig. 9-1(*b*). This is called an *end-bearing pile.* To understand its action, assume that it is driven easily through the mud and silt to the gravel, where it is stopped abruptly by strong resistance, then a heavy load P is applied. This load will try to push the pile downward. The soft mud will probably compress so easily that it offers little resistance to downward motion of the pile. The silt will develop skin friction, but this soil, too, is so relatively compressible that it will not provide a large resistance unless the settlement of the pile is appreciable. The pile cannot settle unless its end is shoved farther into the gravel or unless the gravel compresses. Neither of these two can be done without offering large resistance. It is obvious, therefore, that the greatest part of the reaction to the load P will be under the tip of the pile, whereas some small amount may be applied as frictional resistance on the surfaces of the portion of the pile in the silt. Basically, the pile is to be classified as an *end-bearing* one.

Obviously, the pile in (*b*) should be strong as a column, and its tip should not crush. Such a pile may well have a uniform cross section throughout its length to give it strength as a column and to provide a large bearing area at its bottom.

Piles need lateral as well as vertical support. If the pile of Fig. 9-1(*b*) is long, it may buckle. This buckling cannot happen unless the pile shoves aside the surrounding soil. Even a moderately dense mud or a weak silt will offer sufficient resistance to lateral displacement to prevent serious sideward buckling and bending. This statement is based upon experience and upon the theory that the force required to prevent initial buckling is so small that even weak soils are able to offer sufficient resistance to lateral displacement to prevent buckling of the pile. If the piles pass through water and their tips are embedded little or none in a dense stratum, as illustrated by the gravel alone in Sketch (*b*), the piles will tip over easily, and they must act like long columns.

In practice, an engineer encounters a great variety of conditions. One is pictured in Fig. 9-1(*c*). Here the pile is primarily a friction pile, but the resistance is confined to the lower portion only so that the part above F is still a column. If the stratum above F were soft clay underlain by silt and then by medium sand, the last would cause relatively large resistance so that the load will be withstood by a combination of friction and end bearing. It really makes little difference to the superstructure how the pile obtains its resistance; however, it makes considerable difference to the pile as a member of the substructure.

The area of the surface of a pile that may be relied upon to resist skin friction is usually its actual embedded surface. However, this should not be greater than the minimum area of slippage, whether along the pile itself or along a weaker shear plane in the soil. Figure 9-1 (*d*) illustrates two cases. The dependable area is the minimum area that will circumscribe the pile.

Assume that the pile in Fig. 9-1 (*c*) is driven, then earth fill is added around it to consolidate the mud. The weight of this fill will undoubtedly compress the mud and the silt, and it may compress the clay somewhat. Furthermore, the fill and mud particularly may continue to consolidate for several years. The effect upon the pile is settlement along with the consolidation of the silt and clay because this friction pile must move with the material in which it is driven. The greater settlement of the mud and fill will probably cause a downward frictional force on the portion of the pile above F, this being equivalent to adding to the load P. Therefore, the fill may cause greater penetration or settlement of the pile.

If fill is also added around the pile in Fig. 9-1(*b*), the silt may be compressed, but the gravel is not likely to deform appreciably. The result will be that considerable downward frictional forces may act on the portion of the pile that is in the mud, fill, and silt and cause the pile to deliver a larger concentrated pressure at its tip unless it can penetrate the gravel still farther.

When large lateral loads are to be resisted, a friction pile like that in Fig. 9-1(*a*) will bear against the adjacent soil and act somewhat like a cantilevered beam, as pictured in Sketch (*e*). The character of the distribution of the pressures against the soil is shown also. When the lateral forces are large and when the unsupported top portion of the pile is long, batter piles are desirable, as shown in Sketch (*f*). As the horizontal force H tries to tip the structure toward the left in the picture, all the piles bend somewhat, but the batter pile at the left tends to press downward, whereas the one at the right tends to pull out. The direct resistances along the axes of these two piles have components that oppose H, making them more effective for this purpose than vertical piles would be. Of course, if the tension to be developed in any pile exceeds the dead load on it, means should be provided to anchor the pile effectively into the cap. The holding-down power of UT may be questionable unless the movement of the cap is large. It is also obvious that vertical loads on RS alone would tend to force the superstructure to the right, whereas the actions of RS and UT in a symmetrical arrangement counteract each other for vertical loading.

Assume that a pile has been driven down to a stratum of sand and gravel, as pictured in Fig. 9-2(*a*). When a load P of short duration is applied, the end bearing of the pile transmits it to the sand; the latter may spread it over sufficient area of the underlying silt and clay to prevent failure. However, if the load is continued, the compressible materials in the vicinity of S may consolidate as shown by the dotted line. The sand stratum may not be strong enough to act like a structural slab to bridge over such an area but must conform to the shape of the underlying layer, and settlement of the sand and pile occur at R to equal and accompany that at S.

Consider further the question of the action at S of Fig. 9-2(a). The settlement will probably be caused by the squeezing out of some water from the compressible soil. The overlying sand is porous so that this water can escape. If this soil were a stiff dense clay, it would be relatively impervious, then the water could not escape so easily, and serious settlement might not occur. One should remember that water trapped in a filled bottle will resist tremendous compression. However, he should also be sure that the bottle cannot leak and that no one can smash it.

Dependence upon this principle of trapping the plastic layer may or may not be wise. Can one be sure that the piles will not break up the overlying

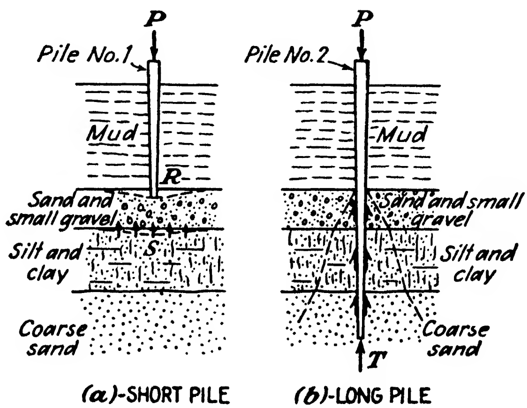


FIG. 9-2. An illustration of action of an intervening hard stratum.

stratum or serve as potential leakage channels? Is there an outlet for the water from S somewhere else through a weak portion of the upper layer? Will neighbors build a structure or drive a well that changes the entire situation?

A second pile is shown in Fig. 9-2(b). Of course it is longer and it costs more than the other, but it is driven clear through the upper layer of sand and the compressible soil to a stratum of deep coarse sand. The question of harmful settlement is thus eliminated. Is this pile almost entirely an end-bearing one with a reaction at T, or is it supported in part by skin friction along the embedded portion below the mud? Initially, the latter is probably the case, but one can see how a slow compaction of the silt and clay might occur, with the result that more and more of the reaction will be at T unless settlement of the pile occurs, too, so that it continues to apply some load to the upper materials. The spreading of the load is pictured qualitatively by the dotted lines in the sketch.

9-3. Action of a group of piles under load. How will several piles act if they are driven near together and all are subjected to simultaneous vertical loading? If the piles are end-bearing ones, as pictured in Fig. 9-1(*b*), each will act like a column and deliver its load directly to the gravel. If the size, material, length, straightness, and penetration of the piles are the same, each may be assumed to carry the same load. If one pile can support safely a load of 30 tons, 10 piles can support 300 tons. This situation does not apply, however, in many other cases.

A group of three friction piles is shown in Fig. 9-3(*a*), and the spreading of load by the cone of distribution of each one as an individual is pictured

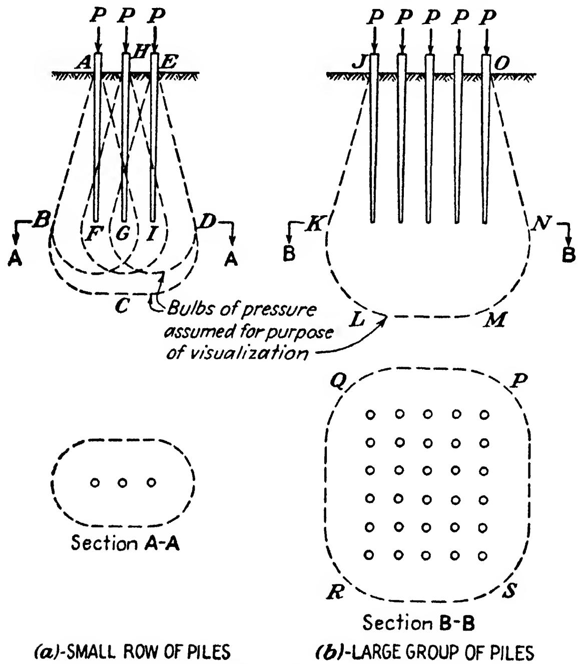


FIG. 9-3. Action of group of friction piles.

by the dotted lines. Obviously, the affected volumes of soil overlap so that material near *F* is affected by the load from two piles; and *G* by three loads. The soil is what it is, and it probably will compress more under the heavier loads than under the lighter ones. As pile *HG* tends to settle more than *AF* and *EI*, load will be shifted from *HG* to the others because all will ordinarily be forced to settle equally if they are under a single cap. The general result may then be pictured as though the load affects a volume represented by *ABCDE*, and the soil under the area shown in section *A*-*A* must resist this force. Since the angle *BAF* depends upon the properties of the soil, the action of the pile group cannot increase it appreciably above that applying for the case of a heavily loaded single pile. Therefore, the safe resistance of this group of three piles should not be considered equal to three times the safe load upon one pile alone because there is less soil supporting the group than there would be under three individual and widely separated piles.

The preceding statements assume that the soil is compressible silt, clay, or very fine sand. Medium and coarse sands, unless underlain by compressible materials, do not have this quality of progressive settlement referred to. However, they are not generally so weak that piles are needed in the first place, unless the soils are under so much water or weak soils that the piles are used as columns or stilts.

In the action of a large group of piles like that shown in Fig. 9-3(*b*), the overlapping of the cones of distribution and the areas of soil affected by one pile are still more apparent. The total safe load of the group is no more than the bearing capacity of the soil below some such area as *PQRS* of section *B-B*. Therefore, the careless use of data regarding the safe load carried by a single test pile, or even by a small group of piles, in the design of large groups of friction piles is likely to be dangerous; so is similar use of data obtained from the resistance of individual piles when they are being driven. In such cases, it is best to study the structure as a whole, including the soil below the piles as a part of the structure. The need for adequate evidence regarding the properties of this soil is obvious. Many piles are used in such a case as that of Sketch (*b*) because the load is large and the soil is weak, yet the situation is one in which the resistance of each pile may be relatively small and unpredictable. Thus, with the greater importance of the structure and the greater difficulty of supporting it may go greater uncertainty regarding the safe bearing resistance per pile, and greater need for sound engineering judgment to attain safety without unwarranted cost. It may be that fewer piles at wider spacing would be just as safe as many piles jammed closely together since the load-resisting capacity of the underlying soil may be the weakest part of the system.

In Fig. 9-3(*b*), the piles are pictured as being loaded equally. This may not be true; those at the periphery of the group may have more resistance per pile than those near the center, but, in cohesive and compressible soils and with a properly designed cap structure, the stiffness and strength of the latter will compel all piles to settle practically the same amount so that, as the load tends to shift from the central to the outer piles, the outer ones bear more heavily on the soil and tend to compress it more or penetrate farther into it. However, considering the other uncertainties involved, it is generally satisfactory to assume that uniformly applied loads on a group of piles, such as in Sketch (*b*), are resisted equally by each pile. It is the magnitude of the safe load per pile that is uncertain. This, of course, does not apply when the loads are eccentric, the character and lengths of the piles considerably different, and the soils nonuniform in any important respect.

Another situation to consider when planning foundations with large groups of piles may be illustrated by Fig. 9-2. Assume that the first pile is driven into the sand and gravel stratum or crust but not through it. The shearing resistance of the crust may spread the load over a considerable area S of the weaker stratum below it, as explained before. This may be satisfactory for one or two piles, whereas a large group designed for the same load per pile may overload the crust, cause the substratum to compact excessively, or compel the weak material to flow laterally. On the other hand, if the piles are driven through the crust and the weak stratum, then well into the coarse sand as in (b), the piles will support most of the load by end bearing, and the structure will probably be satisfactory.

FIG. 9-4. Driving Raymond step-taper piles for the extension of a power plant. The soil is approximately 30 ft. of silt over fine sand. *(Courtesy of Westcott & Mapes, Inc., Consulting Engineers, New Haven, Conn.)*

A rather broadly interpreted analogy of the action of the crust in the vicinity of R of Fig. 9-2(a) may be the following: Assume that this sand stratum is like a thin layer of ice floating on a pond. A boy skates on the ice, and it bends down until it displaces enough water to counteract his weight, provided it has sufficient strength as a structural slab to do so without rupture. Now along come two or three more boys who join him, temporarily. No longer can the ice bend sufficiently and transfer enough forces to support all these concentrated loads; hence there is an accident.

9-4. Pile driving. By pile driving is meant the operation of getting a pile into the position that it is to occupy as a part of the completed substructure. Pounding it down may or may not be a part of the process, although it generally is. The details of the method to be employed, or the combination of methods, will generally depend upon the type of pile, the characteristics of the soil into which it is to be placed, the conditions under which the field operations must be performed, and the equipment that is available for and best suited to the job. Some of the most prevalent methods are described here.

1. Drop hammer. This is one of the oldest methods and is relatively elementary. Somewhat like Fig. 9-4, the equipment consists of a tall framework with a heavy weight or hammer that can slide up and down between guides, means for lifting the pile into position between the leads or guides ready for driving, a cable system running over a top sheave and thence to the drum portion of a hoisting engine or other power unit, and a clamp and trigger arrangement that permits attachment of the cable to the hammer for lifting and then automatic tripping or disconnection of the weight at some desired height from which it will drop onto the top of the pile.

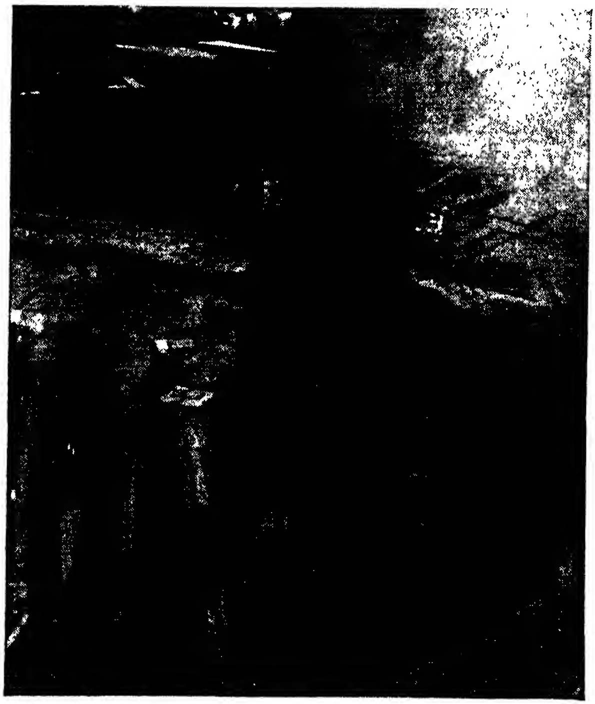


FIG. 9-5. Industrial steam crane and Vulcan no. 1 hammer driving 14-in. monotube piles at pier 35 to penetration of 34 ft. Chesapeake City Bridge, Chesapeake City, Md. *(Courtesy of Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y.)*

The operation of such a hammer requires considerable time between blows, and each blow may deliver a violent shock to the pile. Perhaps unusually long piles cannot be driven well at first if the top (butt) end is so high that there is insufficient distance left for the fall of the hammer. Widely spaced heavy blows may tend to damage the butt or the tip of the pile, or to buckle it sidewise. Between blows, the pile is at rest. When a pile has not been driven so far as the operator believes it should go, there is the possibility that he will raise the hammer far too high and thus damage the pile by too severe a blow.

2. Steam hammer. Figure 9-5 shows a steam hammer in operation. Generally, the hammer itself consists of a short steel frame with guides on which a movable weight can travel. This weight is attached to a piston and cylinder device that operates under steam (or air) pressure and has proper valves so as to lift the weight, then not only drop it but accelerate it still further by steam pressure on the opposite side of the piston. The remainder of the equipment must have means for lifting a pile into position, guides for the steam-hammer unit, and means for raising and lowering the latter.

A steam hammer delivers short rapid blows that tend to keep the pile more nearly in continuous motion and vibration. It delivers a practically uniform blow for any given steam pressure. It is placed upon the butt of the pile and follows the latter down. Such a hammer is less likely to damage a pile than is a drop hammer, and it may cause less violent vibrations of the soil. Outside the corner of an old wall-bearing factory in New England, the blows from a drop hammer caused vibrations of the sand that consolidated the latter enough to cause such subsidence of the soil that the corner of the old structure dropped more than ½ in., with serious cracking of the walls. The designers had intended to use a steam hammer. Whether the same settlement would have occurred with a steam hammer is questionable but not probable.

3. Water jet. This method of sinking piles is illustrated in principle in Fig. 9-6. It consists primarily of a pipe inside the pile or alongside it through which water can be forced under high pressure. This stream of water at or near the tip of the pile tends to loosen and wash out the soil so that the pile can settle easily into or be driven without difficulty through

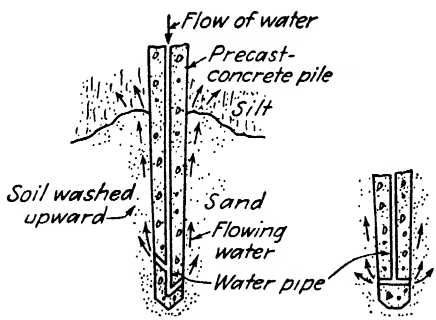


FIG. 9-6. One arrangement illustrating sinking of a pile by means of a water jet.

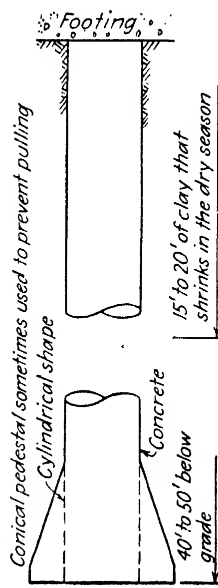


FIG. 9-7. Principle of drilled concrete piles used by Texas Highway Department to prevent movement caused by clay that shrinks when dried and expands when wetted.

the ground to the desired position. It is usually best to drive down the pile a little to "seat" it into final position after jetting.

Naturally, a water jet will turn clay into mud, and silt or rock flour into a semiliquid. This method is therefore most useful when a crust must be penetrated, as in Fig. 9-2, and other methods are, or are thought to be, inadequate or undesirable. It is useful, too, when piles are to be driven into a gravel stratum, as shown in Fig. 9-1(*b*). Of course, this method is used primarily with end-bearing piles because the water disturbs the surrounding soil so much. Granular soils, when displaced this way, will settle back around the pile fairly well; clays and silts may remain disturbed and *lubricated* for many years.

4. Drilling. Figure 9-7 shows in principle a method that has been used by the Texas Highway Department. In the summer the clay will dry out, shrink, and crack considerably. When rains come, the water penetrates quickly and deeply and causes swelling and lubrication of the soil. By "carving" out a hole through the clay down to firm material, using a casing if the presence of intermediate sand or water-bearing strata require it, and filling the hole with concrete, one may construct an end-bearing pile. By means of special equipment the Highway Department has had these holes undercut so as to form a pedestal or sort of footing that is very helpful. It is advisable to extend the piles deeply enough to make sure that skin friction along the part of the pile that is in the area of swelling clay will not lift the pile up. Perhaps this pile is a sort of caisson rather than a true pile.

A modification of this general idea was used to penetrate a new fill containing scraps of rock mixed with a physically unstable volcanic excavated material. A large churn drill was used inside a pipe casing, as for the drilling of deep wells. The hole was drilled down to firm material, cleaned out, and concreted to form an end-bearing pile.

5. Jacking. When a pile is to be driven inside an existing structure where headroom for ordinary pile-driving equipment is not available, it may be possible to force the pile down by using a hydraulic jack that reacts against a heavy weight above it. Of course, this is a highly special procedure and generally requires that the pile be jacked down in short pieces. Short lengths of steel pipes that can be welded together as needed are useful for this purpose.

When a solid pile is driven into a granular soil, it will displace some of the sand and may cause consolidation of the soil because of the vibration, as stated previously. If the soil is plastic and cohesive, *e.g.*, clay, it will probably be forced to move sideward and upward, the latter being the line of least resistance.

Assume that a single pile is being driven into a thick layer of clay, as shown in Fig. 9-8(*a*). As the pile displaces the clay so quickly, little of the water can be squeezed out of it, whereas the clay must be displaced by the pile. The arrows picture the general directions that the clay will probably take, and the lines ACD and EFB indicate a new surface to which the original dotted surface AB may be forced. This may not be harmful for a single pile, for a small group of piles, or for a narrow row of them. It may, however, cause the lifting of previously driven piles of a large group.

For example, assume that several piles represented by JK and LM of Fig. 9-8(b) have been driven through clay into sand as shown. Then another series of piles represented by GH is driven close to them. It is probable that these last ones will cause much of the clay around JK to move upward slightly so that JK is lifted some distance d. Pile LM may be

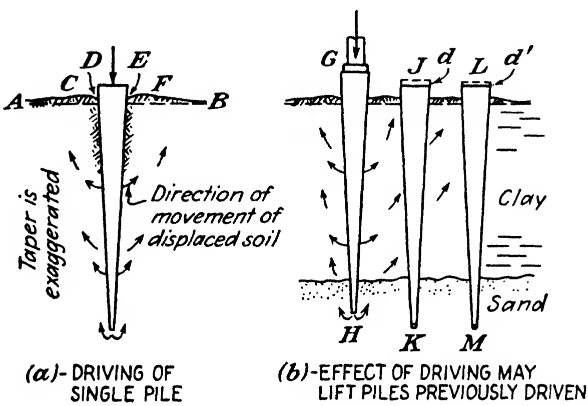


FIG. 9-8. Exaggerated picture of displacement of soil caused by driving of solid piles.

lifted a smaller amount d'. This is likely to weaken their end bearing. Under such conditions, redriving of the piles may be advisable in order to ensure proper bearing. The movement of the soil above the sand may tilt the piles or injure them if the sideward movement of the clay is sufficient and if the bottom ends of the piles are held laterally by the sand. This is a feature to be guarded against in the field. Of course, any construction that merely removes the soil in the space taken up by the pile avoids this uplift and lateral movement.

As a pile is driven, the adjacent soil is greatly disturbed. Sand will adjust itself quickly; clay may be remolded almost to the state of lubrication so that some time is required for it to reconsolidate. However, even in the case of clays, a pile that may drive easily, when put down, may appear to have the clay "set" around it so that it is difficult to start the pile down again after one or several days of rest whereas, after it is started, it may be driven easily once more.

Sometimes it is specified that piles are to be driven to "refusal." This is generally assumed to mean that they are to be driven until the resistance to driving and the driving force are in equilibrium or as near so as they can be without damage to the pile. Driving to such extremes is hazardous; the piles may be injured permanently. It is difficult to determine what should be the stopping point to ensure the maximum resistance to load without damage from overdriving. As stated so well by Fred D. Hartford[1] ". . . an engineer sometimes consents to the public flogging of an innocent pile that certainly does his profession no credit."

9-5. Wooden piles.[2] Centuries ago men used the trunks of trees as piles to hold up water-front and other structures. These products of nature are still used, although now in competition with other types of pile. For the purpose of convenient comparison, the characteristics, advantages, and disadvantages of wooden piles and other types will be discussed under headings arranged in the same sequence but in separate articles.

1. Availability. Trees are a renewable resource and can be grown in many parts of the world However, a tree that is suitable for a large pile may require 75 to 100 years (or more) for growth. The people of the United States have begun to awake to the long-term advisability of curbing the waste of our timber resources and of conducting planned measures for their replenishment.

It is becoming difficult to obtain wooden piles that are sufficiently long and straight. Lengths of 30 to 50 or 60 ft. are reasonably available in pine; those over 60 ft. are readily available in the West Coast area. Wooden piles from 60 to 115 ft. long are readily obtainable in Douglas fir in the Pacific Northwest.

2. Shipment. Wooden piles are relatively light and easy to handle. Since they float, transportation by rafting may be very helpful in the case of water-front structures. Transportation by ship may be made economically over long distances; by railroad, over fairly long distances.

3. Handling. Their lightness and strength as beams facilitate the handling of wooden piles. Their tensile strength also permits them to be pulled readily and reused as parts of construction accessories such as trestles, falsework, fenders, piers, and working platforms. The ease with which they can be cut and worked in the field is also a great asset.

4. Driving. Hard driving is likely to "broom" the butts of wooden piles, and the tips may be crushed when driven through or into dense gravelly materials. The lack of straightness may cause them to deflect from the intended position when driven, and hard driving may crumple them so that yielding may be mistaken for further penetration. One such case is pictured in Fig. 9-9.

5. Adjustability of length. It is very easy to cut off a wooden pile that does not penetrate to the expected depth. When it proves to be too short,

[1] Notes on Driving Timber Piles, *Engineering News-Record*, Nov. 29, 1945.

[2] See also American Society of Civil Engineers, Timber Piles and Construction Timbers, *Bulletin* 17, Manuals of Engineering Practice, 1939.

it can be driven farther by means of a follower, a short piece of timber placed on top of the pile. The permanent extension may then be made of concrete or a short length of wooden pile spliced on the lower one. Wooden piles may also be cut off at a specified elevation under water by means of a saw supported upon a framework above water level.

6. Strength. Wooden piles are good as friction piles; they are not so desirable as end-bearing ones because the compressive strength of the wood is relatively small, the tips in bearing are small also (unless the pile is driven with butt down, a difficult and unusual procedure), and they are not inherently strong columns. Ordinarily, a wooden pile should not have to

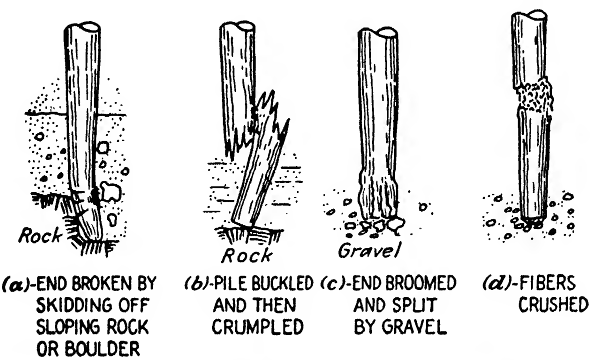


FIG. 9-9. Some types of failure of wooden piles from overdriving after hitting an obstruction.

support more than 30 tons under the best conditions. They can be braced by bolting on diagonal members and struts above or below water.

7. Flexibility. Wooden piles are relatively flexible and springy laterally. They will deflect considerably, offer strong lateral resistance, and spring back into position again, thus cushioning the shock against a colliding ship. This is a great asset in such structures as small piers, ferry slips, fenders, and dolphins. For such service, it is difficult to find a satisfactory substitute for them.

8. Durability. This is one of the grave questions connected with the use of wooden piles. Their usefulness may be destroyed by fungi, marine borers, and mechanical action.

FUNGI. These—a low form of plant life—cause what is ordinarily termed *rotting*. For growth, fungi need food, air, moisture, and a moderate temperature. The food is found in the wood itself. If the piles are continuously immersed in water, the necessary air is excluded. If the wood is perpetually and thoroughly dry, the requisite moisture is missing. If the temperature is frigid or very high, the fungi cannot survive. The conditions for their growth are generally most advantageous at or near the surface of the ground.

Inasmuch as piles are a part of some foundation, and the necessity for them usually arises because of the presence of water and of weak wet plastic soils, about the only means of preventing fungus growths is to have them completely immersed, or to add poison to the wood so as to kill the organisms or greatly retard their growth.

This matter of submerging the piles below the "permanent" watertable may seem easy and the results trustworthy. However, it is fraught with dangers in many cases, and extremely good engineering judgment is needed to determine the wise course of action. Two illustrations are the following:[1]

1. The Boston Public Library on Copley Square, Boston, was built on untreated wooden piles. Some years later settlement was found to be caused by decayed piles It is reported that the ground-water level had been lowered, probably as the result of building and draining a subway.

2. In the Brooklyn (N.Y.) Navy Yard, buildings which had been standing safely for 40 years began to settle. The untreated wooden foundation piles were found decayed below the original ground-water table. The original water level had been materially lowered by pumping in the western end of Long Island, which had been going on at an increasing rate to meet industrial and other demands. In some area the ground-water level was below tide level, but inflow of salt water was prevented by the mud cover on the shore.

One of the best methods for preserving wooden piles through poisoning of the fungi is pressure creosoting of the piles. In the full-cell process, a vacuum is created around the piles, then the creosote is applied under pressure. This is best for marine piles and timbers. The empty-cell process for foundation piles and other timbers utilizes initial air, then the application of the preservative under pressure, and finally the application of the vacuum. Paint coatings and surface creosoting are inadequate. The wood should be as completely impregnated as possible. Bolt holes, notches, cutoff of butts, and even shrinkage cracks may provide access for the fungi to any untreated portion of the wood. If possible, all fabrication should be done before creosoting—but this is not always practicable. However, field applications should be used to protect such cuts as much as possible. Proper creosoting will generally prolong the life of wooden piles many years.

MARINE BORERS. These creatures occur mostly in sea water or brackish water. Among the destructive varieties are the Teredo and Limnoria. The following also is quoted from Pile Foundations and Pile Structures:

The severity of attack varies greatly in different localities. Marine borers are most active in warm waters and in the tropics, but they have done serious damage to timbers in waterfront structures almost to the northern and southern limits of the

[1] American Society of Civil Engineers, Pile Foundations and Pile Structures, *Bulletin* 27, Manuals of Engineering Practice, 1946.

respective temperate zones. Borers are found in Norway, practically in the Arctic, and a dock has been destroyed in 18 months in Alaskan waters, latitude 60°N.

Where marine borers are not present, consideration must be given to the possibility of their introduction. For many years the waters of the upper part of San Francisco Bay, in California, were considered free from marine borers, but a sudden infestation caused the failure of waterfront structures. Boston and other New England harbors (borers first noted in 1933) are other such examples. The water of New York harbor is now practically free of borers—possibly because of the low oxygen content due to sewage pollution. The introduction of sewage treatment on a wide scale in such a harbor may result in the admission of marine borers in destructive quantities.

Since the water conditions which govern attack are not completely understood, a prediction as to the future is uncertain, and conservative practice normally justifies treatment of piles and timbers in marine water. In fresh-water lakes and rivers, marine borers are not present, although in southern streams *Sphaeroma* may be destructive during periods when the water is brackish. *Teredos* are destructive in some fresh-water harbors in India and Australia.

The following data are quoted from "Protection against Marine Borer Attacks," by William G. Atwood:[1]

Many years of research and study have not resulted in finding any method of preservation as economical and effective as impregnation with coal-tar creosote. To use this material properly it is necessary to obtain the best creosote, use enough of it, select the timber having the required strength and other qualities which can be properly treated, and see that the treatment specified is obtained and that the timber is not damaged after treatment.

The most satisfactory timber for treatment is southern yellow pine, especially that with a large amount of sapwood. Heartwood can not be penetrated to any appreciable depth and, therefore, any heart requirement in the grade of timber used should be waived. The U.S. Forest Products Laboratory has proven that there is no difference in strength between heartwood and sapwood, or between the different species of pine. Specifying longleaf timber instead of southern yellow pine is a pure waste of money, and frequently obtains timber with a large amount of heartwood, which is resistant to treatment. The use of standard specifications of the American Society for Testing Materials or of the American Railway Engineering Association will generally result in lower prices and more uniform material.

Considering the cost of the pile alone, a 16-lb. per cubic foot treatment with coal-tar creosote, which is as light as should ever be used for piles in salt water, about doubles the cost of a pile delivered in New York. One of the recently built New York City one-story piers cost about $622,000, of which about $62,000, or 10 per cent, represented the cost of the untreated piles themselves. If these piles had been treated their cost would have been 18 per cent of the total, or an increase of 8 per cent. The two-story piers recently built cost about $933,000, with an untreated pile cost of about 6.6 per cent. If these piles had been treated the pile cost would have been about 12.5 per cent, or an increase of only 5.9 per cent in total cost.

[1] *Wood Preserving News*, December, 1935.

Driving of wooden piles into gravel or soil containing boulders may damage the piles seriously so that the treated portion is crushed or cracked to such an extent that decay of the interior can occur. Even above the soil, hard driving may cause or increase cracks that permit fungi or borers to attack the pile. Therefore, one should be careful to avoid anything that will nullify the benefits of the protective treatment that he has spent so much to obtain.

A brief outline of some of the data given by R. D. Chellis[1] relating to the physical protection of wooden piles against attack by borers is the following:

1. Complete protection is necessary:
   a. Good material and excellent workmanship are essential.
   b. Bark is only temporary protection. Limnoria will enter through knots.
   c. Packed fill is good protection, even riprap may be helpful.
   d. It is essential to seal the bottom in soil. Scour may remove soil relied upon.
   e. Abrasion may remove or damage coverings.
   f. The wooden piles should be creosoted properly even though they are to be covered by some protective coating.
2. Metallic armor:
   a. The surface of a pile must be made as smooth as possible.
   b. Metal may be nailed on in advance over bituminous membrane.
   c. The material of the cover and nails must resist corrosion well. Copper and zinc are costly.
   d. Scupper-nailing with copper nails may resist Limnoria if done properly.
   e. Iron and steel sheathing and mesh are not practicable.
   f. Such coverings are not used frequently now.
3. Concrete encasement:
   a. Metal shields with spacers may be used as forms.
   b. Precast covering sections may be suitable. There may be a question regarding proper sealing.
   c. Encasement poured under water may not be uniform and strong.
   d. Hay process (Cement Gun Co., Inc.). The concrete is placed in a watertight form above water, then both are lowered to position. The form is removed later.
   e. Presscrete method. A metal form is used, the water is ejected, and concrete placed by compressed air, and the form is removed later.
   f. Johnson-Western method. A circular reinforced gunited jacket, 5 ft. long, is made above water with felt over spacer battens as a

[1] Finding and Fighting Marine Borers, *Engineering News-Record*, Apr. 1, 1948.

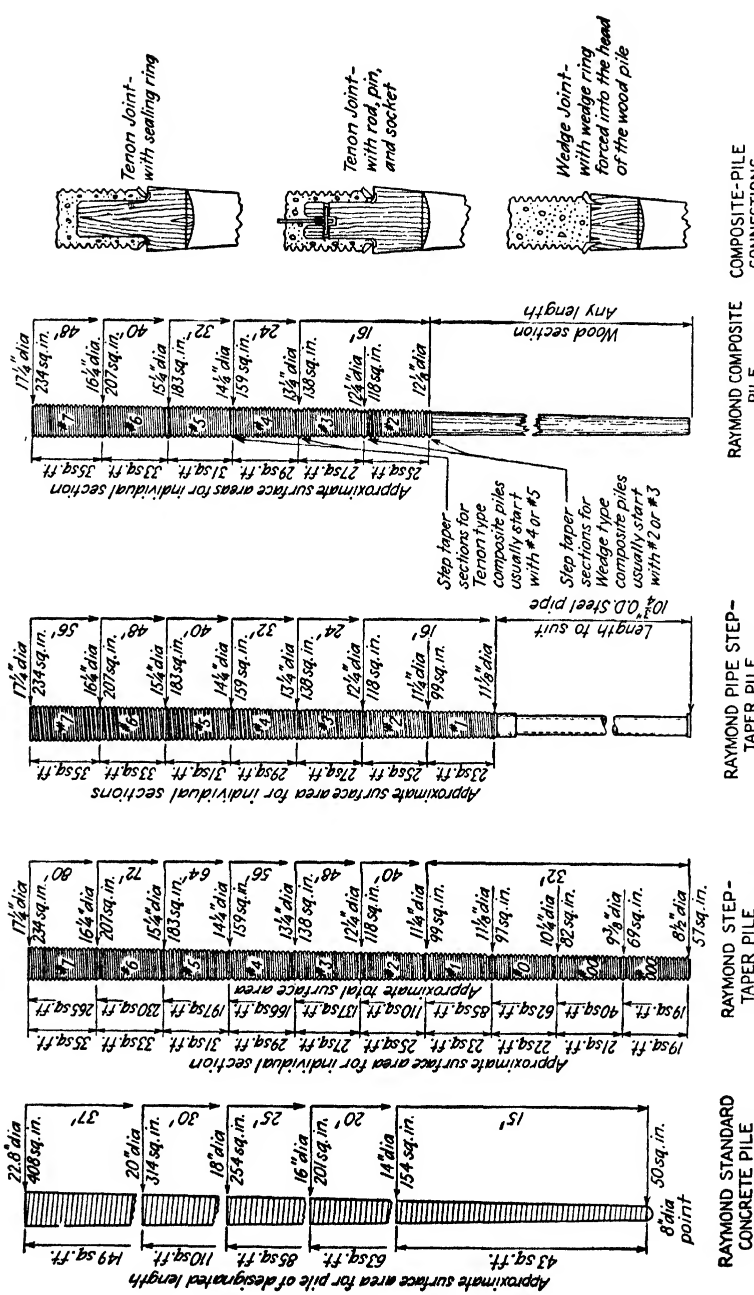


FIG. 9-10. Standard pile dimensions with approximate surface and cross-section areas. (Diameters shown = O.D. of shell.)

back form. This form is lowered into mud, others are placed on top of it, and the annular space is grouted.

g. Shotcrete or gunite may be used to make a reinforced casing put on before driving. Arsenious oxide (As_2O_3) seems to reduce solubility of portland cement. Pressure-jacketed concrete may also be used, this method being controlled by The Presscrete Co., Inc.

Marine borers cannot live below the mud line. They need the supply of oxygen furnished by open water. Their growth might therefore be prevented by encasement of the piles in reinforced concrete or noncorrodible metallic shells. The former is likely to crack to pieces because it is relatively rigid and brittle whereas the wooden pile is flexible; the metallic covering is difficult to apply properly and to protect from damage.

Mechanical action. Wooden piles may be damaged by abrasion caused by ships, ice, and driftwood; also by granular soils that are propelled along the surface of the ground by wind, current, or waves. Piles that are lashed together to serve as fenders may be worn badly by chafing. Fire, too, may destroy wooden piles when they are above water.

9-6. Cast-in-place concrete piles. There are several variations of this general category, but they may be divided into two classes: (1) those having a metallic shell that is driven and left in the ground, then filled with concrete, and (2) those having the shell removed after or during the deposition of the concrete.

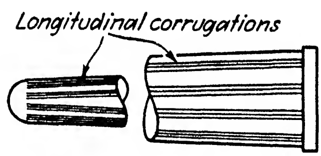


FIG. 9-11. Union Metal monotube thin, steel-shell, concrete-filled piles. (Shell is capable of being driven as a unit.)

In the first class are the following:

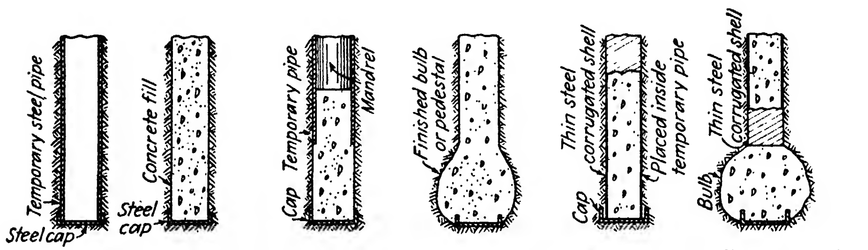
1. The Raymond tapered or step-tapered pile, such as those shown in Fig. 9-10. The shell is driven down by means of a mandrel that is withdrawn after the shell is in its final position. The concrete is then poured. These are primarily friction piles.

2. The Monotube pile shown in Fig. 9-11. These shells are strong enough to be driven as a metallic pile, then they are filled with concrete. These, too, are generally best as friction piles.

3. The MacArthur corrugated-shell pile pictured in Fig. 9-12. A strong open-ended pipe and an inside core are driven first, the core is removed, a corrugated shell is dropped inside the pipe, the shell is filled with concrete, and the pipe is withdrawn. These are primarily end-bearing piles.

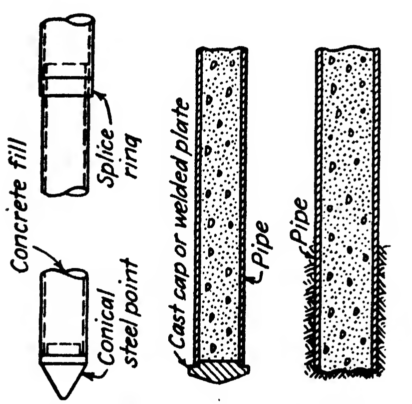
4. The closed-end steel-pipe pile shown in Fig. 9-13(*a*) or (*b*). This is driven into the ground with a steel shoe of some sort or a concrete button to close the bottom end. The pipe is then filled with concrete. These are especially good as end-bearing piles, but they may be of considerable value as friction piles also.

5. The open-end steel-pipe pile pictured in Fig. 9-13(c). It is driven with the bottom end open. This enables the pipe to cut through gravelly crusts and to penetrate to rock or hardpan without displacing the soil as much as would a closed-end pile. The soil inside the pile is then removed by water and compressed-air jets, or by direct excavation, and the pipe is filled with concrete. These are best as end-bearing piles because the excavation methods might impair the frictional resistance of cohesive soils.



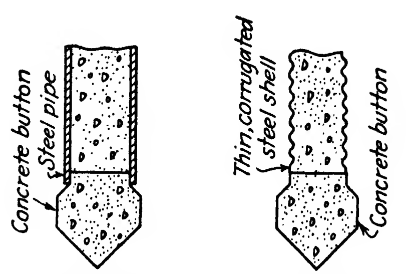
(a)-CAP AND PIPE DRIVEN BY MANDREL (b)-CONCRETE FILL RAMMED DOWN AS PIPE IS WITHDRAWN (c)-INITIAL STAGE IN MAKING SIMPLE PEDESTAL PILE (d)-FINAL PEDESTAL PILE OF PLAIN CONCRETE (e)-PILE WITH THIN STEEL SHELL (f)-PEDESTAL AND THIN STEEL SHELL COMBINED

FIG. 9-12. Some cast-in-place concrete piles made by MacArthur Concrete Pile Corp. (Similar construction is used by others.)



(a)-NATIONAL TUBE CO.'S PIPE PILE (b)-CLOSED-END PILE (c)-OPEN-END PILE

FIG. 9-13. Some steel-pipe, concrete-filled piles.



(a)-DRIVING PIPE LEFT IN PLACE IF NEEDED (Not customary) (b)-THIN SHELL TO HOLD CONCRETE (Driving pipe removed)

FIG. 9-14. Western Foundation Corp.'s button-bottom piles.

6. Another variety of poured-in-place concrete pile is shown in Fig. 9-14. This has been developed by the Western Foundation Corporation and the Western Concrete Pile Company. It consists of a precast-concrete "button" as shown in Sketch (a) that is driven down by means of a heavy detachable pipe. A corrugated shell inside this pile may then be filled with concrete whereas the driving pipe is withdrawn.

These are especially useful for end-bearing piles under conditions where one must penetrate unreliable material overlying a good stratum of dense sand, gravel, or hardpan. There is no doubt as to the reliability and symmetry of the button, and its large area usually provides adequate bearing for heavy loads. Of course, the settlement of the surrounding material into the space disturbed by passage of the button and occupied by the driving pipe renders this type inadvisable for general use as a friction pile. Another advantage, however, is the fact that, by dropping a second driving pipe inside the outer one, the bearing value of the button on the soil can be determined by a load test without having any resistance produced by friction from surrounding soil.

In the second class—piles without metallic shells—are such as the following:

1. The simple cylindrical concrete pile shown in Fig. 9-12(*b*). This may be made by driving a pipe with a closed end or core, filling it with concrete, and then withdrawing the pipe. The wet concrete will settle against the soil as the restraining effect of the pipe is removed, or the concrete may be rammed down. This is especially useful as an end-bearing pile, but it may have considerable value as a friction pile also if there is good contact between the soil and the concrete along the sides.

2. The MacArthur pedestal pile pictured in Fig. 9-12. After the pipe is driven and the core removed, some concrete is deposited in the pipe. The core is reinserted and forced down so as to press the concrete into a bulb or pedestal below the pipe to provide a greatly increased end-bearing area. The pile is then completed as in item No. 1.

The following information gives the author's ideas regarding the advantages and disadvantages of cast-in-place piles:

1. Availability. The metallic shells and pipes are generally available, and the concrete can be secured or made almost anywhere. The materials are trustworthy. The piles without permanent shells require little or no metal.

2. Shipment. The light metallic shells and pipes can be shipped easily; short lengths may be welded together. The Raymond step-tapered pile has parts that are fabricated to fit together so that the shells need not be shipped as long units.

3. Handling. The shells and pipes are so light and strong that handling in the field is relatively fast. The concreting operations are simple and efficient also.

4. Driving. The driving of these piles is generally relatively simple and reliable. Boulders, large gravel, buried objects, and very hard driving may tear thin shells. Jetting is seldom resorted to unless done in advance to loosen the soil. If the crust must be broken through, the mandrel or a pipe may be driven part way and withdrawn before the shell is driven down to final position. These piles can often be inspected reasonably well before concreting by lowering electric lights in the shells. The open-end pipes are especially useful where movement of the soil during the driving of solid piles is feared, *e.g.*, next to existing structures or piles. Pipe piles are also very strong so that they may be the best for driving in bouldery soil without failure. In one job, such piles occasionally deflected off the edge of a boulder so that they actually tilted. However, the pipes were not damaged badly, the questionable piles were detected, and they were pulled out

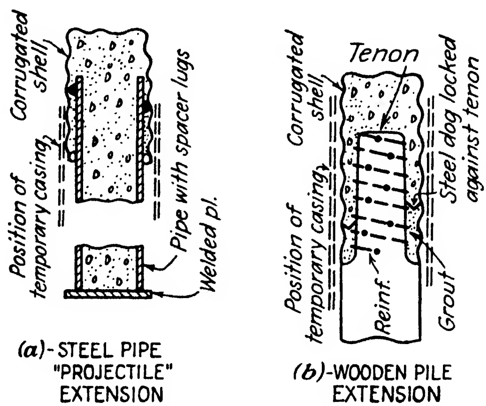


FIG. 9-15. Two types of composite pile used by the Western Foundation Corp.

or not relied upon. The shells and pipes may be used as a permanent part of batter piles.

5. *Adjustability of length.* In this respect these piles have a great advantage. Excess lengths of shells may be burned off, extensions may be welded on unless the total exceeds that permitted by the mandrel or core, the concrete may be poured to the desired elevation, and reinforcement may be used in the concrete sometimes, except in the shell-less type where settlement of the concrete might make this difficult.

6. *Strength.* In most cases the strength of these piles is excellent, if the soils permit its development. The concrete is the main supporting material. However, the following points are raised as questions to be borne in mind:

a. Will the driving of piles alongside newly concreted ones damage the latter? Several empty pile shells should be left between the concreted ones and those being driven.

b. Can the piles be redriven (reseated) if heaving of the ground occurs?

c. Will water inside the shell or pipe cause the concrete to be poor in quality?

d. Will granular soils settle amidst the concrete of shell-less piles as the pipe is withdrawn, causing constrictions or eccentric sections? Will water in the soil weaken the concrete under similar circumstances?

e. Will the bottom of a pedestal pile be shaped as hoped for? Will it be forced to one side to cause an eccentric bearing, and will the displaced soil injure adjacent piles?

These piles are heavy, and their own weight should be included in the loads that are applied to the surrounding and underlying soils.

7. Resiliency. These piles are a rigid type and, except for the pipes, may crack if subjected to lateral bending and shocks.

8. Durability. These piles are inherently durable because of the concrete, unless the soil contains acids. At one industrial plant, the soil near a slag dump was found to contain considerable sulphuric acid. Even though the shells may rust away, the concrete should be durable in other situations.

9-7. Precast concrete piles. In Fig. 9-16 as shown some details of precast concrete piles as presented by A. E. Cummings, Research Engineer of the Raymond Concrete Pile Co.[1] These are strongly reinforced as columns and designed to resist the bending and shocks that may be incident to field operations. The tapered piles are generally limited to the shorter lengths, 35 to 40 ft. The parallel-sided ones are those used for the larger sizes, which may be as much as 24 in. or more in width and over 75 ft. long.

1. Availability. Precast concrete piles may be made of high-strength concrete placed in forms at or near the site of the job. Their reinforcement may be very strong, properly encased, and well tied together. The materials may generally be secured conveniently. Means for adequate curing, handling, and storage must be provided. If there are to be insufficient piles to warrant the provision of necessary forms and equipment at the site, or if there is inadequate space there, then these piles may not always be procured easily.

2. Shipment. These piles are so heavy that transportation is costly except when it can be done by barges. The piles should be cast at or near the location of any large job. The transportation cost may be a great disadvantage if only a few piles are to be used, and it is necessary to set up special equipment to load and unload such long heavy members.

3. Handling. A precast concrete pile 18 in. square and 60 ft. long may weigh approximately 10 tons. This requires heavy equipment for handling, and it generally necessitates a long reach also. The piles must be picked up by slings or special accessories that prevent excessive bending when the piles are raised from a horizontal position. Special strong equipment is needed also for the transportation of the piles about a large site. The pile driver itself must be able to handle these heavy members.

4. Driving. The inertia of a heavy pile tends to resist the shock from a hammer blow. On the other hand, even though the pile is given a relatively small initial velocity by the blow, its momentum is still large, and a

[1] Pile Foundations, reprinted in 1940 from the Proceedings of the Purdue Conference on Soil Mechanics and Its Applications.

correspondingly strong resistance to penetration is necessary to stop it. However, the use of a cushion on the pile head is usually essential to prevent local spalling of the concrete under the impact of the hammer. Otherwise, since these piles are strongly reinforced, they can be driven safely in most cases. If one glances off a boulder, the lower portion may be cracked, or the tip spalled. These piles can be reseated if swelling of the ground makes this necessary. They are also so strong that they are unlikely to be damaged badly by the driving of adjacent piles. The use of a water jet is

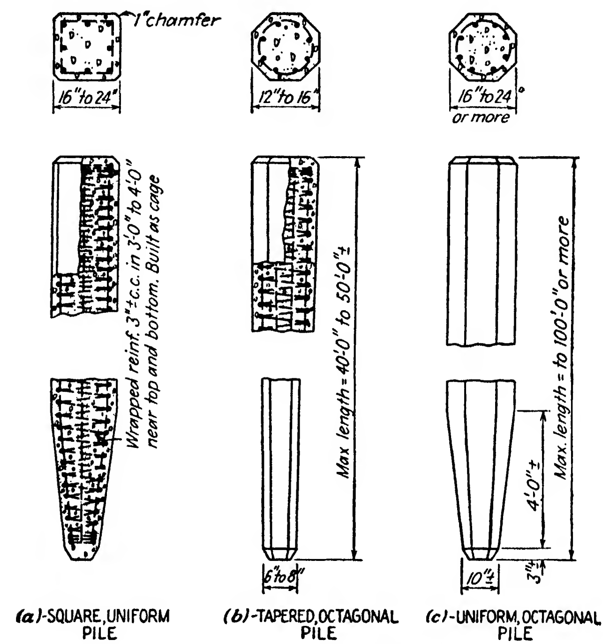


FIG. 9-16. Precast concrete piles.

especially advantageous with precast piles when they are to be driven through or into gravelly materials.

5. Adjustability of length. This is one of the disadvantages of precast concrete piles. If they prove to be too long and cannot be driven to the anticipated depth, it is difficult to cut them off. Even if cut off, the top of the remaining concrete is likely to be cracked. If the piles are too short, it is difficult to extend them so as to secure any effective splice with the original pile. This feature may be especially bad when the piles pass through air or water and are to serve as columns that may be subjected to lateral bending and buckling. An irregular rocky bottom, or one containing boulders, may render it almost impossible to anticipate the proper lengths when the piles are ordered or cast. In such a case it may be possible and desirable to order the piles a few feet short, to leave about 3 ft.

of the longitudinal reinforcement projecting for splicing, to use a special follower that is designed to fit over the rods during driving, and then to cast on them whatever reinforced extensions are necessary.

6. Strength. Precast piles, especially those having a constant cross section, can be excellent columns, and they can be good friction piles, too. They are excellent when needed to resist uplift, provided the reinforcement or the pile as a whole is anchored properly into the main substructure. The tips may have a large bearing area, especially when jetted down. One can be reasonably certain of the strength of such a pile as far as column action is concerned. Another possible advantage of great value is the fact that a precast pile may be subjected to a heavy load immediately after driving. This is very helpful when a trestle is being built and the erection equipment is to build each unit of the structure in series, being supported each time upon the part previously erected. It is a great help also when a large group of precast (or wooden) piles is being driven if the piles can be used promptly to support the pile driver because such equipment cannot "flit" from place to place. Of course, the weight of these piles should be included in the loads applied to the underlying soil.

7. Resiliency. Since they are reinforced so well, these piles are strong in beam action if held properly. However, severe bending as vertical cantilevers projecting from a sandy bottom is likely to cause tensile cracking of the concrete and lead to deterioration. Sharp lateral blows may crack and kink them. They should not be subjected to service that requires true resiliency.

8. Durability. In general, embedded precast concrete piles are very durable. One source of possible deterioration of exposed piles is the spalling of the concrete at and just above the water line because of the freezing of absorbed moisture, the crystallization of salts in the pores, the rusting of reinforcement, and the attack of chemicals in sea water. Acids and other chemicals in the soil may injure them. These chemicals may be caused by leakage from industrial operations and sewers; by leaching from piles of coal, slag, and cinders; and by organic decomposition. Occasionally electrolysis may be harmful. It is not desirable to ground a structure to one or two piles that have steel as a part of their structural system. Ice, driftwood, colliding ships, and wind-blown and wave-washed sand may cause physical damage. Special cements, dense concrete, adequate cover over reinforcement, and bituminous coatings are means of reducing disintegration.

It has been reported that square precast piles used in the bents of a large trestle in sea water had visible shrinkage hair cracks in the portions above the water. This is not improbable in the case of such heavily reinforced members that are not subjected to a sufficiently heavy dead load because the column strength of the longitudinal steel may exceed the tensile strength of the concrete, whereas shrinkage during setting and curing compels the concrete to shorten. Loads that compress the reinforcement would probably close the cracks. Whether these cracks will cause ultimate rusting of the steel and spalling of the concrete remains to be seen. A paint coat of bituminous material or some other suitable one might seal the cracks sufficiently.

9-8. Steel piles. Under the classification of steel piles are included those that depend primarily upon steel for resistance to longitudinal and transverse loads. The pipes and shells used as a part of poured-in-place concrete piles will resist some loads, especially the thicker pipes. If the latter are designed as the chief load-supporting constituent, then these piles might well be classified as steel piles. However, this name is used here generally to denote steel H piles and those in which a steel member is the principal part, whether it is encased in concrete or not.

Among the advantages of steel piles are their great strength as columns, the high unit strength of the metal itself, the long lengths that may be used, the ability of the piles to cut through obstacles and penetrate into the underlying materials, and their strength as beams. When certain of these qualities are necessary, steel piles may be desirable regardless of cost. In Fig. 9-17 are shown some details that may be applicable for work involving steel H piles. These are discussed in the following paragraphs:

1. Availability. Steel piles are procurable in many places; in others, they are hard to get; in times of steel shortage, it may be almost impossible to secure them. This is a matter to consider when a structure is in the planning stage.

2. Shipment. Long heavy steel piles may be shipped by water or rail, although this is likely to be expensive. Hauling them by trucks is likely to be both difficult and costly. Short pieces may be welded together in the field to make long members if necessary, but this is seldom done by preference.

3. Handling. Steel piles are so strong as beams that they can be handled safely if equipment with sufficient load-lifting capacity and reach is available.

4. Driving. These piles are so strong as columns that they will generally withstand any driving forces and impacts that can be applied to them. The cuts A of the flanges shown in Fig. 9-17(a) accentuate the cutting properties of the end of an H pile, but these may not be necessary. They tend to prevent bearing of the tip of a flange on a piece of rock that would cause eccentricity of bearing, and they enable the piles to cut into disintegrated and weak rock more easily. However, the pile may not bear on all of the area at its tip. These plain H piles do not displace much of the soil into which they are driven—sometimes a great advantage. They can be used as batter piles as well as vertical ones. Of course, the driving and handling equipment must be suitable.

5. *Adjustability of length.* Steel piles that prove to be too long can be burned off readily at the proper elevation, even under water, although this may be wasteful. Those that are too short may have scarfed extensions butt-welded to them, or field-welded splices may be used to join the parts. These junctions may be very strong.

6. *Strength.* As indicated previously, the strength of a heavy steel H is tremendous, provided it cannot buckle. The load-carrying ability may be

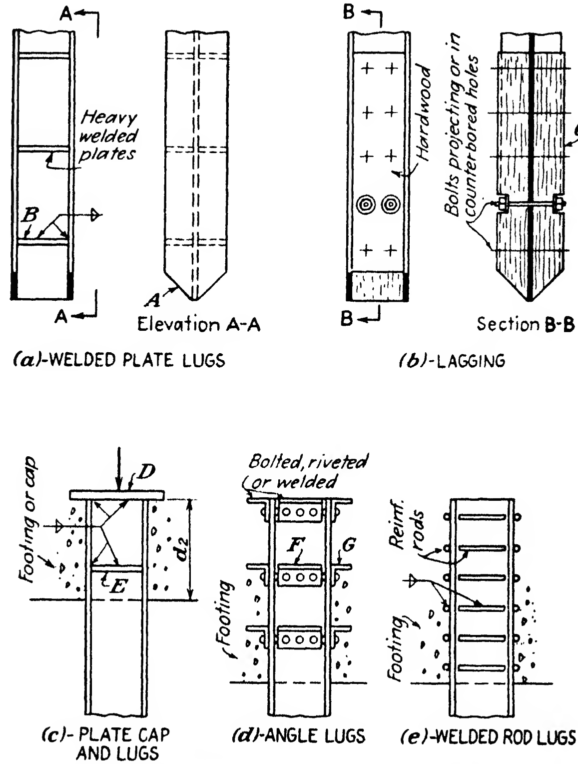


FIG. 9-17. Some details that may be used on steel H piles.

reduced, however, if the load cannot be delivered to the pile properly or if the necessary bearing resistance cannot be developed. In this connection, notice the following points illustrated in the sketches in Fig. 9-17:

(*a*) The welded plates *B*, or a series of them, may be used to increase the effectiveness of the bearing if the pile is driven into sand or gravel. This is intended to provide more resistance than that of friction along the surface of steel only. In the case of gravelly soils, plates *B* may be very effective if they are thick and the welds are strong enough to withstand the driving. The upper plates are of questionable value unless the lower ones fail. Of course, such plates make the driving more difficult.

(b) The hardwood blocks C constitute lagging that is also used to increase the bearing resistance of the pile. This is no stronger than its bolted connection to the steel.

(c) This shows a cap plate D that is welded to a pile to increase the bearing area of the concrete of the footing on top of the steel. The welded plates E are for the same purpose. If the embedment d_2 is 5 or 6 ft., the bond of the concrete may be of real value, but it is not likely to be adequate or absolutely trustworthy for loads of such large magnitude. Grease, dirt, or paint may weaken the bond. The bearing of the concrete directly upon the end of the H is limited by the crushing strength of the concrete.

It may not always be practicable to use details like those shown in Fig. 9-17 at the ends of steel H piles, especially if they are driven to irregular rock so that their tops must be burned off in the field, or if they are driven to varying and unpredictable elevations under water. Tests made by the Ohio Department of Highways[1] indicate that, if the thickness of the concrete footing above the ends of the piles is 2 or 3 ft., there is great resistance of the concrete against penetration of the end of the plain pile without any cap plates. Of course, the footing should be massive and reinforced to prevent splitting and cracking. The bearing on the end of the pile is most useful; the bond along its side may be helpful but, as stated previously, it should not be trusted unless the embedment is several feet. On the other hand, one must be sure that the intended load can be delivered to the piles without any uncertainty.

(d) Here angles F and G may be riveted or welded to both the web and the flanges of the pile. The use of very wide outstanding legs on these angles is ineffective because they have little shearing resistance except near the connnected leg. It is better to use a series of narrow but thick angles closely spaced.

(e) This sketch shows a series of welded pieces of reinforcing rod that serve as lugs. The amount of welding is probably large compared to the bearing area obtained. The concrete footing in any case must be strong enough in bearing and in shear to transmit the required load to the pile.

7. *Resiliency.* A steel pile should not be called resilient; it is strong and elastic. Large lateral deflections may cause overstressing and permanent deformation of the steel. Nevertheless, such a pile will not crack and shatter. Even though bent or somewhat kinked, it may still support a large load.

8. *Durability.* The question of the durability of an unencased steel pile frequently causes argument. Here are some comments upon the subject:

a. Steel piles completely embedded in dense clay will probably be permanent. Rusting requires the continued presence of a supply of oxygen. When the soil is nearly impermeable, the water will not circulate through

[1] Research Report No. 1, Ohio Department of Highways, Dec. 1, 1947.

it to any appreciable extent. As soon as the oxygen in the available water is used up, corrosion will be stopped.

b. In porous soils, the ground water may be moving slowly so that the supply of oxygen is replenished. Some rusting may then occur.

c. Soils and moving water containing acids or other injurious chemicals may cause serious corrosion. Bituminous coatings may be helpful if they can be applied and are not damaged.

d. Sea water is likely to be very injurious.

e. Fresh water and air may cause moderate rusting.

f. Electrolysis may cause serious trouble if it can occur.

g. The sand-blast effect of water-borne or wind-blown sand may damage exposed steel close to the ground.

h. Encasement with concrete is beneficial if the concrete is dense, of good quality, and properly reinforced so that it cannot break off.

In general, steel piles have a very important place in engineering construction. When one needs very long strong end-bearing piles, this type may be the best that can be found. A few such piles supporting 50 tons or more apiece may be far more desirable than a large number of weaker piles, provided the steel will be durable.

9-9. Sheet piles. Some types of steel sheet piling are pictured in Fig. 9-18. As indicated by their name, these are designed for the special purpose of forming a wall or sheet which, when supported properly, will resist lateral pressures. They might be of some value in supporting vertical loads, but this is not an efficient use of the material in them. The joints are made so that, when one pile is driven, the next one can be attached to the edge of the first and be compelled to remain attached thereto during and after the driving. One can easily realize the importance of this feature; without it, one could not expect to drive the piling accurately enough to form a suitable wall.

In Sketch (*a*) is shown a slender type of sheet piling suitable only for small depths and light pressures, unless it is braced at frequent intervals.

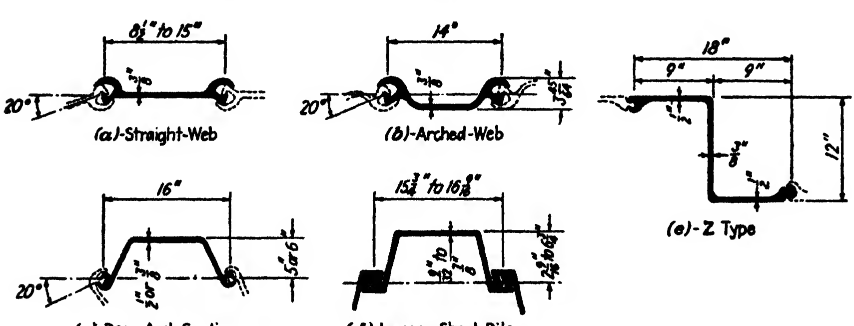


FIG. 9-18. Illustrations of some types of steel sheet piles.

Sketches (*b*) and (*c*) show designs for stiffer piling made to span much longer distances between supports. Sketches (*d*) and (*e*) show other strong types. The drawings indicate how the edges are locked together so that they can slide with respect to each other but cannot separate. The use of these members will be illustrated in connection with cofferdams and bulkheads.

For shallow depths and small jobs, wooden sheet piling may be made of planks, as shown in Fig. 9-19(*a*). It is intended to be driven to form a locked wall as indicated in (*b*), but it is difficult to keep the parts tight. For example, assume that member *A* of Sketch (*c*) has been put in place, then *B* is driven at a slight angle or is forced away by jamming of soil in the groove so that space *C* occurs near the bottom. There is little that can be done to prevent this. Perhaps bevels at the bottom as shown in (*d*) would cause *B* to crowd against *A*. This might or might not happen.

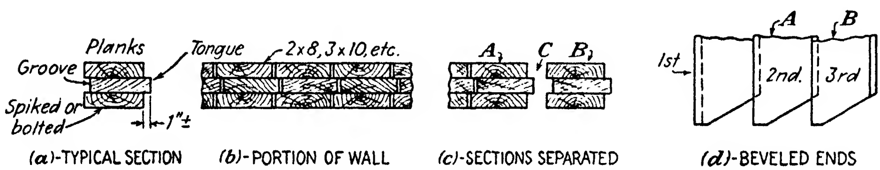


FIG. 9-19. Some details of wooden sheet piling.

Sheet piling is very important as a material to be used in the construction of foundations. It is seldom a permanent structural element of a substructure, *i.e.*, one upon which the safety of the superstructure depends after the structural work is completed. Many times, the sheet piling may be left in place wholely or in part as a construction expedient or necessity but not to support loads except for such cases as the following:

1. To form an enclosure that would trap a mass of fine sand that the designer fears may otherwise become quick and flow out from under the load.

2. To hold back all or part of the side of an embankment or other earth mass on which a structure is or is to be supported.

3. When used first as an enclosure around an excavation, then bonded to or with the tops buried in a concrete base so that the piling will trap the soil inside and also support such vertical loads as it may.

9-10. Pile-driving formulas. A pile-driving formula is an attempt to evaluate the resistance of a pile to the dynamic forces applied upon it during the driving and to estimate from this the static longitudinal load that the pile can support safely as a part of the permanent substructure. The use of such a dynamic formula generally involves knowledge of certain properties and characteristics of the equipment and the pile, together with the measurement or estimation of various data in the field. It is logical and reasonable to expect that a pile that is driven down easily will support less load than an identical pile that offers considerable resistance before it is "seated." Furthermore, it is natural that an engineer should desire to have some means of determining in advance what load may be placed safely upon a pile. Otherwise, how can he tell how to plan a specific foundation?

In general, a pile-driving formula endeavors to take into account and utilize the following:

1. The magnitude of the impact or energy delivered to the pile by the blow of the hammer. This may be expressed as the mass of a drop hammer, or the mass of the moving part of a steam hammer, times one-half the square of the velocity that it has as it hits the pile.

2. The magnitude of the work done per blow of the hammer. This is supposed to be the product of the resistance overcome and the distance that the pile is moved by the blow. The latter is often taken as the average penetration per blow made during the last ten blows.

3. The determination and separation of the losses, inertia forces, and any other nonusable resistances from the total resistance so that the remainder may give the magnitude of the real resistance of the pile to driving and, it is hoped, to future longitudinal static loads. The doing of this presents many questions.

Among the uncertainties that beset the user of any pile-driving formula are these:

1. The resistance of the hoisting rope, drum, and guides may reduce the velocity of a drop hammer so that its energy at the instant of impact is not Wh, the weight times the height of fall. The operator who checks the movement of the rope just before the hammer strikes the pile reduces the blow, too.

2. Rebound of the hammer may add to the effectiveness of the blow so that it exceeds Wh. The secondary blow after rebound may or may not assist the driving by occurring in proper phase with the vibration of the pile.

3. Incorrect steam (or air) pressure, friction in the stuffing box, leakage of steam, and improper operation of the valves of a steam hammer may affect the energy of the blow considerably compared to the rating stated by the manufacturer.

4. The inertia of the pile must be overcome. Just how much of the energy of the blow is required to do this?

5. The resistance of the soil to displacement and heaving is difficult or perhaps impossible to ascertain, and the compaction caused by the local decrease of voids in the soil is almost certainly unknown.

6. The work dissipated as heat, the effect of a cushion or block on top of the pile, and crushing of the material of the pile may be of real consequence.

7. Temporary lubrication of the surface of the pile because of water squeezed out of the soil may cause the pile to penetrate the ground more easily than it would after being at rest for some time.

8. Except for truly end-bearing piles, the resistance to driving offered by an individual pile cannot provide the engineer with a real value of the reduction of the safe load that may be necessary because of the group effect —often of tremendous importance.

9. The slow compaction of the surrounding and underlying soils and their tendency to flow from under localized loads are not revealed. The driving is too rapid to reveal this except as it may yield evidence of serious plastic yielding.

10. Measurements taken in the field may not be accurate. Probably the reader would not do any better himself.

The formulas in Table 9-2 are presented for whatever benefit they may give. However, this is done with the warning that they are to be used with caution.

TABLE 9-2. PILE-DRIVING FORMULAS*

J. F. Redtenbacher:

$$R_d = \frac{AE}{L}\left[-s + \sqrt{s^2 + \frac{2W^2hL}{EA(W+P)}}\right] \qquad \text{Formula I}$$

A. Hiley:

$$R_d = \frac{AE}{L}\left[-(s+C) \pm \sqrt{(s+C)^2 + \frac{2L}{AE}eWh\frac{W+n^2P}{W+P}}\right] \qquad \text{Formula II}$$

Engineering News formula for steam hammers:

$$R = \frac{2Wh}{s+0.1} \qquad \text{Formula III}(a)$$

Engineering News formula for drop hammers:

$$R = \frac{2Wh}{s+1} \qquad \text{Formula III}(b)$$

Modified *Engineering News* formula:

$$R = \frac{2Wh}{s+0.1(P/W)} \qquad \text{Formula IV}$$

Pacific Coast formula:

$$R_d = \frac{AE}{2L}\left[-s \pm \sqrt{s^2 + 4Wh\frac{(W+n^2P)}{(W+P)}\frac{L}{AE}}\right] \qquad \text{Formula V}$$

Karl Terzaghi:

$$R_d = \frac{AE}{L}\left[-s \pm \sqrt{s^2 + 2Wh\frac{(W+n^2P)}{(W+P)}\frac{L}{AE}}\right] \qquad \text{Formula VI}$$

* These are quoted directly from Pile Foundations and Pile Structures, *Bull.* 27, published in 1946 by the American Society of Civil Engineers.

In formulas I, II, V, and VI, R_d is the ultimate load and h and s must be expressed in inches, *i.e.*, in the same units. In formulas III(a), III(b), and IV, R is the safe load, with a factor of safety of 6; h is in feet and s is in inches. The meanings of the symbols used in these formulas are the following:

Symbol	*Definition*	*Usual unit*
W	Weight of striking parts of hammer	Pounds
P	(1) Weight of pile as driven; (2) total load	Pounds
h	Height of fall of hammer	Feet (or inches)
s	Penetration of pile per blow	Inches
E	Modulus of elasticity of pile as driven	Pounds and inches
C	Temporary elastic compression of the soil	Inches
A	Cross-sectional area of pile as driven	Square inches
L	Length of pile as driven	Feet (or inches)
n	Coefficient of restitution (Newtonian)	
e	Efficiency of hammer	Per cent
Rd	Dynamic resistance of soil (ultimate load or bearing capacity of pile)	Pounds (or tons)
R	Allowable static safe load on pile	Pounds (or tons)

The many uncertainties show that an engineer should look upon the results computed from the preceding formulas as one piece of evidence, but only an indication of what *may* be the allowable bearing value of a pile. The results of the test loads and computed values shown in Table 9-3 are extremely instructive. They tend to frighten one because they show that the results obtained from various formulas range from dangerous to wasteful, and no one formula seems to yield consistent values. As a matter of curiosity, the scores were averaged for the cases shown, and the results recorded at the bottom of the table. This is done to show the reader that such averages may mean little; a great variation from the average is what may indicate disaster or waste.

As for other cases involving uncertainties, the safety factor used in conservative designing may come into play to save one's structure. Nevertheless, one should not depend upon it too much.

9-11. Load tests. At first, the reader might believe that a load test of one or a few piles will remove all uncertainties regarding the safe bearing value of the piles to be used in a large job, provided that all are the same as those tested, and that all are driven by the same equipment, under similar conditions, to equal depth, and to equal penetration per blow. Load tests are exceedingly helpful. However, the results should be examined carefully before they are depended upon and before conclusions are drawn because of the following reasons:

1. The soil conditions throughout the site may not be the same as those at the locations of the test piles.

2. The tests may yield few or no data that enable one to estimate the reduction of safe load that should be applied to the results of the tests because of the group effect of the large number of piles to be used near together in the actual substructure.

3. The test loads are applied for such a short time that no data are given regarding the slow compaction and flow of the soil that may occur because of the future long-term loading of the area.

TABLE 9-3. Ultimate Loads by Various Formulas Compared with Actual Static Test Loads at Failure

Actual loads at failure, tons	Formula I (Redtenbacher)		Formula II (Hiley)		Formula III(a) (*Engineering News*)		Formula IV (modified *Engineering News*)		Formula V (Pacific Coast)		Formula VI (Terzaghi)			
											$n^2 = 0.25$		$n^2 = 0.1$	
	Tons[a]	%[b]	Tons[a]	%[b]	Tons[a]	%[b]	Tons[a]	%[b]	Tons[a]	%[b]	Tons[a]	%[b]	Tons[a]	%[b]
1. Timber Piles; Single-acting Steam Hammer and Cohesive Soils[d]														
93.0	108.0	116	67.0	72	270.0	290	.....	...	82.5	89	113.0	122	109.0	117
81.0	104.0	128	131.5	162	169.8	210	.....	...	85.7	106	109.0	135	106.0	131
70.0	43.5	62	37.0	53	75.0	107	.....	...	40.0	57	50.3	72	46.0	66
66.5	62.0	93	58.0	87	106.0	159	.....	...	60.0	90	73.0	109	69.0	103
96.5	47.0	49	41.0	42	88.0	91	.....	...	44.0	46	54.3	56	50.0	52
100.0	73.0	73	68.5	68	180.0	180	.....	...	60.8	61	81.8	82	77.5	77
110.0	33.5	30	29.0	26	56.0	51	.....	...	32.8	30	38.4	35	35.5	32
70.0	43.5	62	37.0	53	75.0	107	.....	...	40.0	57	50.8	72	46.0	66
2. Concrete Piles; Single-acting Steam Hammer and Cohesive Soils[d]														
153.0	155.0	101	124.0	81	648.0	423	375.0	245	161.0	105	247.5	162	195.0	127
172.0	151.5	88	122.5	71	480.0	279	357.6	207	160.5	93	228.0	132	182.0	106
234.0	240.0	103	189.5	81	1,482.0	630	507.0	216	211.5	90	348.0	148	287.5	123
87.5	100.0	114	75.5	86	....	....	214.5	244	120.0	137	191.0	218	138.0	157
127.5	130.0	102	96.0	75	....	....	171.0	134	130.0	102	238.5	187	179.0	141
117.5	184.0	156	229.5	195	....	....	399.0	339	176.0	150	273.0	232	222.5	189
117.5	71.5	61	60.5	51	....	....	208.5	178	89.5	76	127.5	108	91.0	77
117.5	84.0	71	67.5	57	....	....	228.0	194	103.5	88	152.5	130	111.0	94
210.0	153.5	73	118.0	56	....	....	252.0	120	150.0	71	266.0	127	205.0	98
210.0	200.0	95	153.0	73	....	....	330.0	157	183.0	87	315.0	150	252.5	120
210.0	188.5	90	145.0	69	....	....	306.0	146	175.0	83	304.0	145	241.0	115

3. Steel H-beam Piles; Various Hammers and Soil Types

203[c d e]	154.0	76	149.5	73	375.0	184	.....	...	139.0	68	180.5	89	165.0	81
105[c d]	70.5	67	81.0	77	222.0	211	192.0	183	69.5	66	88.0	84	76.5	73
100[c d]	41.0	41	40.5	40	45.5	45	.....	...	44.0	44	49.0	49	44.5	44
50[c d]	45.6	91	43.5	87	69.6	139	.....	...	48.0	96	51.6	103	48.3	97
150[c d]	121.0	81	127.5	85	....	....	327.0	218	111.0	74	151.0	100	127.5	85
80[c f]	82.0	103	78.0	98	129.0	161	.....	...	81.0	101	89.0	111	85.0	106
70[d g]	41.5	59	44.0	63	85.8	122	79.2	113	48.3	69	54.0	77	46.8	67
50[d g]	46.0	92	44.5	89	72.0	144	.....	...	49.4	99	53.1	106	49.3	99
40[d g]	78.0	195	78.5	196	127.2	318	.....	...	76.5	191	89.0	222	83.0	207
81[f h]	151.0	186	116.5	144	90.0[h]	111[h]	.....	...	135.9	167	159.5	196	255.0	315
69[f h]	241.5	350	110.5	160	166.8[h]	241[h]	.....	...	236.0	341	255.0	370	246.5	357

4. Average of Values

From Part 1

86	64	77	59	70	127	149	.....	...	56	67	71	85	67	80

From Part 2

160	151	96	126	81	870	444	304	198	151	98	244	158	191	122

From Part 3

91	97	122	83	102	138	168	199	171	94	120	111	137	112	139

Parts 1, 2, and 3 are taken from Pile Foundations and Pile Structures, *Bull. 27*, American Society of Civil Engineers, 1946. From discussion by G. G. Greulich of Pile Driving Formulas, Progress Report of the Committee on the Bearing Value of Pile Foundations, *Proceedings*, American Society of Civil Engineers, September, 1941, Table 2, p. 1393.

[a] Ultimate load at failure (not the safe load) computed by the formula indicated.

[b] Percentage of the test load at failure. Failure load is the load that produced increase of settlement disproportionate to increase in pile load.

[c] Single-acting steam hammer. [d] Cohesive soil. Noncohesive soils would not usually show so great variations as cohesive soils. [e] Not the maximum load.

[f] Noncohesive soil. [g] Double-acting steam hammer. [h] These tests were by drop hammer, Formula III(*b*).

4. The settlement of the test piles to be used as that constituting practical failure is difficult to establish without being chosen arbitrarily. Perhaps the applicable building code specifies it, but that does not mean that it is correct.

5. It may be necessary to set some average load as that to be considered to cause this critical settlement if the data obtained from the tests differ.

6. A safety factor should be applied to the *ultimate* bearing capacity of the piles as determined by the tests. Perhaps this should be 2, perhaps something larger, or something smaller. Excellent engineering judgment is needed in making this decision.

7. It is probable that the more nearly the piles act in end bearing, the more reliable the results of the tests will be, unless the tips of the piles are underlain by plastic or other compressible materials, or soils that may flow.

To be preferred are pile tests made in advance of the planning of the structure. Then, besides information indicating load-bearing capacity, these tests may yield valuable data regarding the best type of pile for a particular site, the length of pile needed, the pile-driving equipment required, and whether jetting is necessary. All these will probably enable the designers to develop a safe and economical design for the structure. However, it is sometimes difficult to convince an inexperienced owner that the money spent for such tests is a good investment. Too often he believes that the cost of getting equipment to the site, driving piles that may not be a part of the permanent structure, and making the tests is wasted and unnecessary. He seldom pays attention to the additional expenditures that he does not see—those covered in the construction costs of a design that is and should be made very conservatively because of the lack of proper information. Furthermore, he will complain vehemently if the engineer guesses wrong and designs a structure that is unsatisfactory.

When jobs are small and when pile-driving equipment cannot be obtained in advance, it may be necessary to use the data obtained from borings and soil tests for the selection of the type, length, and bearing value of the piles used for design purposes. Then arrangements should be made in the contract—on a unit-price or force-account basis—for the testing of a few of the first piles driven. This will enable the engineer to obtain data on which he can base the instructions for driving and penetration, and he can check his assumptions in time to revise the design if he finds that he has been too optimistic or too pessimistic in his ideas about the loads that the piles may support safely.

Figure 9-20 shows one method of making a load-bearing test of a pile. A few points to be considered are the following:

1. The loading platform should be strongly built.

2. The pile should be steadied laterally by bracing that will not participate in the resistance to vertical loading.

3. Settlement readings should be made by taking levels upon some point or points of the platform which are continuously accessible and which yield data giving the actual settlement of the pile, not deformation of the platform, tilting of the pile, or movement of the soil. A proper bench mark should be used.

4. The load should be applied so that its magnitude may be determined accurately. Weighed metallic or concrete blocks, bags of sand, and members of structural steel may be suitable. Water in a tank may serve the purpose.

5. The load should be applied in predetermined, and preferably uniform, increments. For piles that may be expected to support safely only 15 or

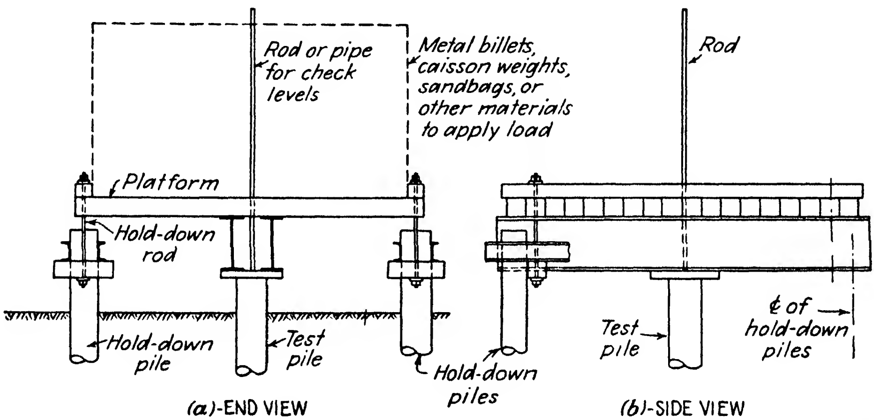


FIG. 9-20. One arrangement for a loading platform for making static load test of a pile. (Adjustable hold-down rods to anchor piles prevent overturning. Rods must be slack and balance maintained when settlement is measured.)

20 tons, the increment may be 5 tons; for much stronger piles, it may be 10 tons. The use of a large increment may shorten the duration of a test but, when the critical load is approached and when the last large increment produces a settlement that is called failure, one may not be sure that the pile was not previously almost at the point of yielding, so that only 2 or 3 tons were necessary to constitute the "last straw." The application of a safety factor of 1.5 to this final test load may yield a result that is on the side of danger.

6. Each increment of loading should remain on the test pile until settlement for a period of 1 or 2 days is negligible, except when the load proves to be the critical one, and serious, perhaps continued, settlement occurs.

7. Settlement readings should be taken before and after the application of any load increment, and probably at 6-hr. intervals between loadings. The results should be watched closely and checked carefully, especially when inconsistencies seem to occur.

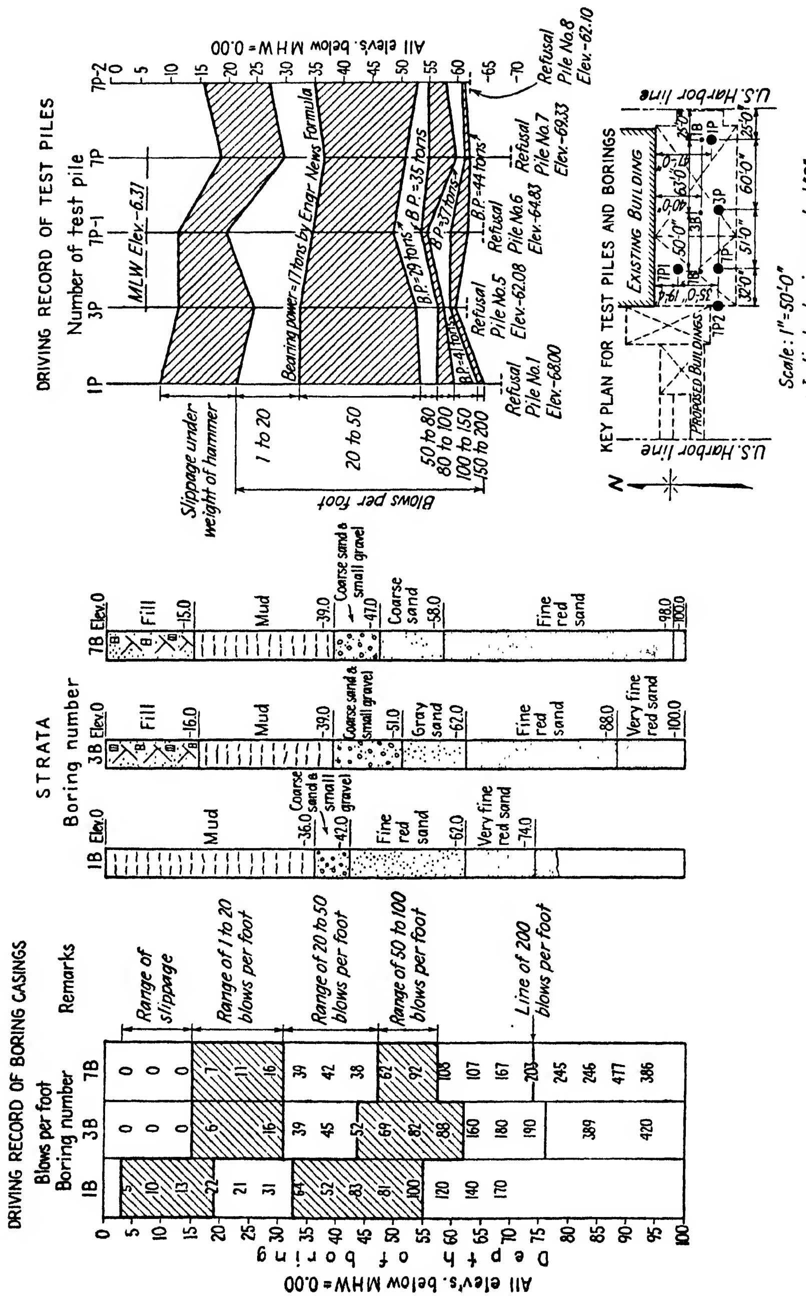


FIG. 9-21. Record of borings and pile tests made for an extension of a power plant. (Courtesy of Westcott & Mapes, Inc., New Haven, Conn.)

8. It is important also to obtain settlement or rebound readings and the corresponding loading, as the test load is removed. The final reading after the load is off will enable one to determine the permanent settlement of the pile.

9. Those who are making the test should observe and record all other data that may have any probable usefulness when the results of the tests are studied. Among these, are such things as the number of blows per foot of penetration for the complete driving.

Naturally, such a load test as that previously described takes time. When necessary, some useful data may be obtained from a more rapid test. However, a rapid load test is likely to yield reliable information in cases where the piles reach granular soils and where they are largely end-bearing ones. These tests may not provide good information on the safe loads for friction piles in plastic cohesive soils. It is obvious that group effect and

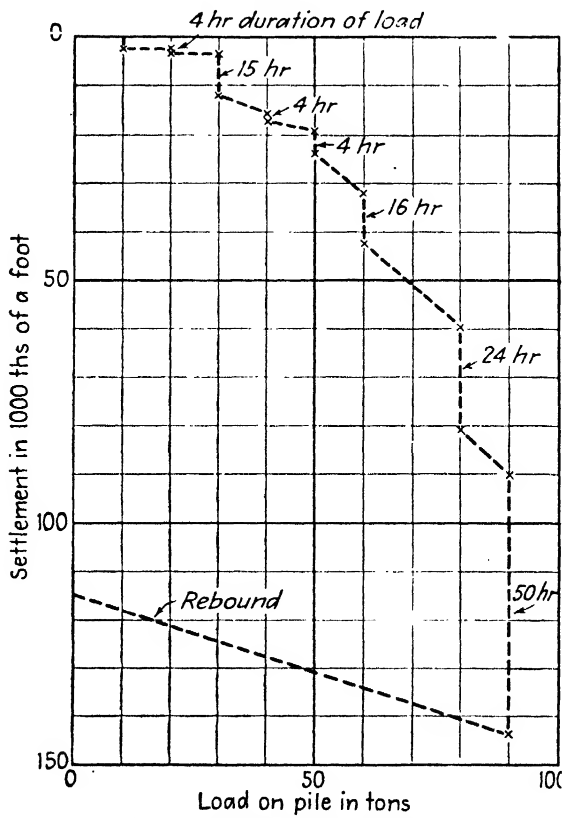
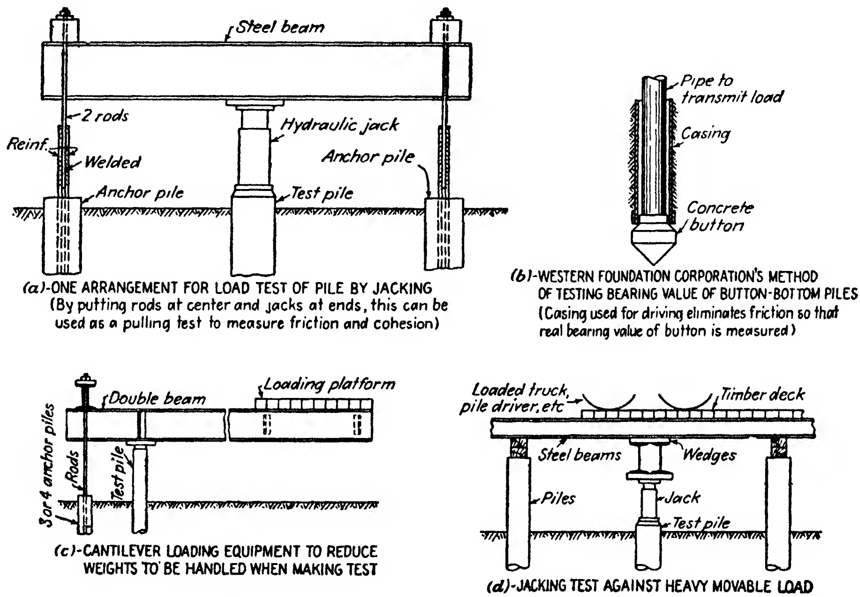


FIG. 9-22. Load-settlement record of a pile tested by Westcott & Mapes, New Haven, Conn. (Length 36 ft. 7 in. Ten blows per foot for last foot. Pile driven through silt into thin gravel stratum above fine sand. Raymond step-taper pile with larger section welded to end to increase end diameter to 11 in. Net settlement 0.115 ft.)

probable compaction of soil are revealed even less than they are by the longer tests. Nevertheless, the latter are so poor in this respect that the rapid tests may be as accurate as other uncertainties justify.

One method of making a rapid load test of a pile is indicated in Fig. 9-23. This assumes that a row of piles has been driven for the permanent foundation. A heavy beam is attached to one or two of the piles on each side of the one to be tested. A powerful hydraulic jack is then placed between the pile top and the beam. Pressure is applied to the pile by increased amounts



(a)-ONE ARRANGEMENT FOR LOAD TEST OF PILE BY JACKING (By putting rods at center and jacks at ends, this can be used as a pulling test to measure friction and cohesion)

(b)-WESTERN FOUNDATION CORPORATION'S METHOD OF TESTING BEARING VALUE OF BUTTON-BOTTOM PILES (Casing used for driving eliminates friction so that real bearing value of button is measured)

(c)-CANTILEVER LOADING EQUIPMENT TO REDUCE WEIGHTS TO BE HANDLED WHEN MAKING TEST

(d)-JACKING TEST AGAINST HEAVY MOVABLE LOAD

FIG. 9-23. Miscellaneous loading devices for testing the bearing value of piles.

at short intervals. By means of the gage pressure and level readings taken on the top of the pile, the load-settlement curve may be plotted. Certain questions should be raised concerning this procedure:

1. It tests primarily the frictional and shearing resistance along the pile's surface and in the soil between the tested pile and those that anchor the beam. No reliable measure of soil compaction is found because no extra load is added to the area.

2. When the piles are primarily end bearing, there may not be enough resistance to uplift to prevent pulling of the anchor piles.

3. The anchor piles should not be too close to the test pile, perhaps 5 or 6 ft. from it.

4. There may be difficulty in attaching the beam to the anchor piles unless the former can be bolted or welded to steel H's, shells, or anchored reinforcement.

When piles are to resist uplift, a pulling test may be made by attaching a beam to the top of the test pile, then using jacks under the ends of the beam to apply upward pressure. Here again one should be careful how he extrapolates the result of a test on one pile to a large group because the latter can be no better than the weight of the soil that anchors them.

Another variation of making such a test is the building of a platform on which a heavily loaded truck or the pile driver itself may be placed. A jack may then be placed on top of the pile so that it can react against the

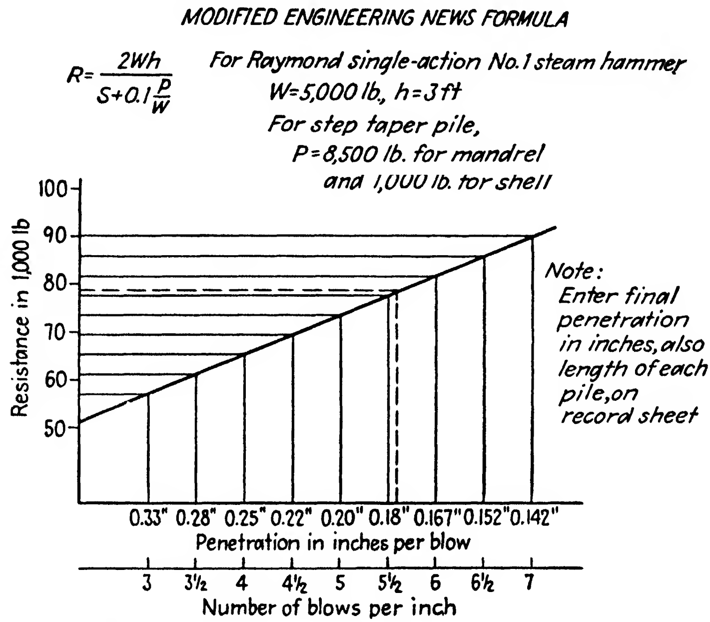


FIG. 9-24. Example of a reference chart for field use during pile-driving operations when a particular formula is used as a guide.

load on the platform. One must be sure, however, that he can obtain accurate readings of the settlement of the test pile.

In general, load tests of piles are so small compared to the forces applied to a structure that they aid little in estimating the settlement to be expected from the compaction of plastic soils around and below a large group of piles. Nevertheless, if the test pile does not support the desired load safely, it is a reasonably sure bet that the average bearing value of the group of piles will be even less, and perhaps very much so.

The effect of vibrations upon the safe bearing value and settlement of piles is another source of uncertainty when one attempts to interpret the results of static load tests. Probably this feature is negligible in the case of end-bearing piles but not for friction piles. Tests made for the General

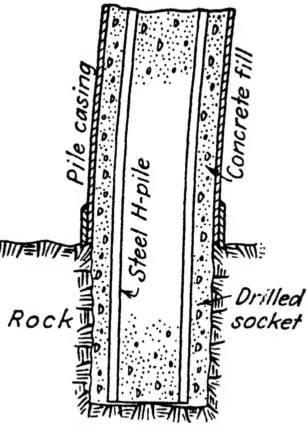


FIG. 9-25. Drilled-in pile or caisson for very heavy loads. *(After data of Western Foundation Corp.)*

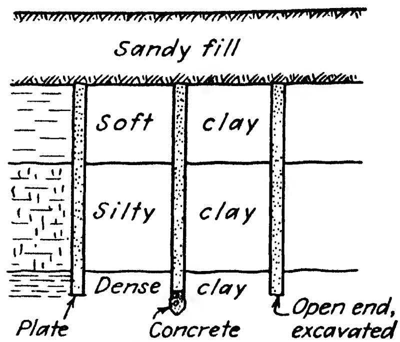
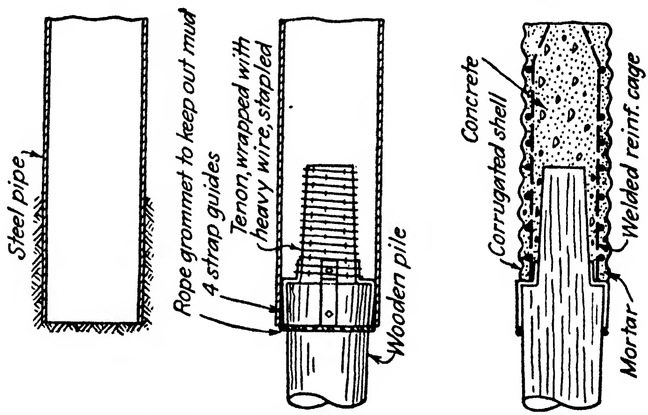


FIG. 9-26. Three ways of making sand-pile drains by driving steel pipes, filling with sand or gravel, and withdrawing pipe.

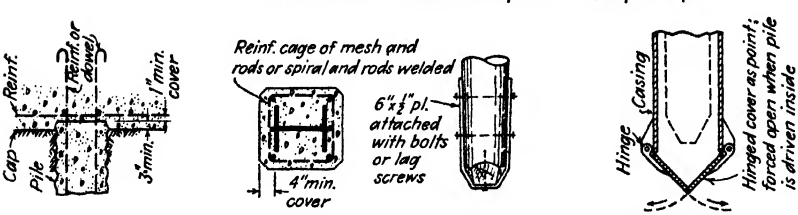


(a)-PIPE CASING DRIVEN WITH CORE INSIDE; CORE REMOVED

(b)-WOODEN PILE DRIVEN INSIDE CASING

(c)-COMPLETED SPLICE; CASING REMOVED

FIG. 9-27. Construction and details of a splice in a composite pile.



(a)-ATTACHMENT OF CONCRETE PILE TO FOOTING

(b)-ENCASEMENT OF EXPOSED PORTION OF STEEL H-PILE

(c)-PLATE TO PROTECT WOODEN PILE AGAINST BROOMING DURING DRIVING

(d)-RECTANGULAR CASING USED BY WESTERN CONCRETE PILE CO. TO BE DRIVEN PART WAY TO ENABLE EXTRA LONG PILE TO BE LOWERED INSIDE, DRIVEN, AND CASING REMOVED

FIG. 9-28. Miscellaneous details.

Electric Company at Schenectady, N.Y.,[1] indicate that vibrations increased the settlement of heavily loaded friction piles that were driven into the following strata: approximately 30 ft. of old fill; a soft organic sandy clay; and rather uniformly graded sandy silt and silty clay, together with some compressible fresh-water clay. The increased settlement caused by vibrations under working loads is probably considerably less than that of test piles loaded to capacity. However, the group effect in the compaction of granular materials supporting many piles that are all affected by vibrations simultaneously may be serious. This problem should be considered when one is planning pile foundations for generators, heavy machinery that is subject to shocks, railroad trestles and bridges when the live loads are relatively large, and very heavy cranes when settlements caused by them may be harmful to the structure in general or to costly equipment nearby.

With rapid tests as well as with slow ones, somebody must determine what constitutes failure of the piles, what is to be taken as the critical load, and what the safety factor should be for design purposes. When one considers the empirical nature of these things as well as their great importance, he will begin to realize the character of some of the decisions that must be made in the practice of foundation engineering, and the responsibility that someone must assume.

[1] C. F. Dodge and W. F. Swiger, Vibration Testing of Friction Piles, *Engineering News-Record*, May 13, 1948.

10

PILE FOUNDATIONS

10-1. Introduction. This chapter contains discussions and illustrations of general principles and procedures that are likely to apply to pile foundations. There are many other considerations and details that are dependent largely upon the special conditions at a given site and the requirements of any particular structure. Many of these will be included in subsequent chapters.

It is impossible in a single chapter to tell all that should be known about the planning and designing of pile foundations. Different engineers use different methods, as is to be expected. The illustrations, however, should give the reader some help in inventing a plan of attack of his own upon his special problems.

10-2. Choice of type of pile. In the planning of a pile foundation, one of the first decisions concerns the type of pile to be used. Several types may be practicable, but the problem is the selection of the best one. Then this type is generally used in the design and specified in the contract unless alternate proposals are asked for in the hope of securing lower bid prices.

Each problem is a specific one—a given structure, with given loads and dimensions, at a particular site, and to be supported upon the soils that exist at that place. Knowing these things, how can one determine the best type of pile to use?

The advantages and disadvantages of various types may be judged to a certain extent from the data given in the preceding chapter. At least, it may be possible to eliminate a few that are obviously less suited to the conditions than are others. A list of things that may influence the decision is the following:

1. Length required
2. Bearing value desired per pile
3. Accessibility of site
4. Means for handling piles
5. Materials on hand or easily available
6. Elevation of watertable with respect to pile caps

7. Presence of marine borers
8. Comparative costs
9. Degree of permanence required
10. Experience with construction used in other similar structures
11. Ease of installation
12. Speed with which piles can be secured
13. Speed of installation
14. Piles best suited to methods and conditions of driving
15. Possibility of damage to piles after driving
16. Adaptability to varying lengths if found necessary
17. Ease of cutting off or otherwise securing proper elevations of tops
18. Ease of increasing number of piles if necessary
19. Personal preference of the engineer or owner because of past experience with a particular type
20. Availability of a contractor and his equipment able to install a certain type
21. Desire to utilize services and products of a particular contractor for business reasons and because of favorable experiences with his work in the past
22. Driving above or below water level
23. Flexibility or resistance to bending required
24. Value of a pile as a column if end bearing
25. Area of tip of pile if end bearing
26. Resistance of pile to hard driving when this is necessary
27. Piles previously used if new work is an extension of an old structure
28. Minimum disturbance of soil if this is a factor
29. Ease of driving at an angle when batter piles are needed
30. Presence of acids or other materials in the soil that would injure certain types

If the type of pile is left to be settled after the receipt of competitive proposals, there may be many and heated arguments in arriving at a decision. Each contractor or manufacturer believes in his own product, and each may guarantee that his piles will support the specified loads safely. Judgment based upon price alone may not be advisable. Each type of pile has special advantages for certain uses. If this were not so, competition would eliminate many of them. On the other hand, a contractor engaged in foundation work usually installs various types of pile in the course of his work, and he can do so readily. It is desirable for the engineer to decide for himself in advance which type is best, obtaining counsel in this matter when he needs it. After that, the competition is secured through the desire of various contractors to get the job.

10-3. Allowable bearing value of a pile. The determination of the allowable (or presumptive) bearing value of a particular type of pile to be used under the specific conditions existing at a site is a very serious but uncertain matter. Stating a figure is one thing; making this figure the correct one is something else. This may as well be admitted.

An engineer may tell his men to use 65-ft. thin-shell poured-in-place concrete piles for a particular structure, and to allow 30 tons per pile. The men then will go ahead wholeheartedly with their design work because of their confidence in him. Later, a chosen pile-driving formula in which are applied the data obtained during driving of piles at the site may yield a computed safe load of 35 tons each for these piles. Then everyone is still happy because of confidence in the formula.

The selection of the allowable load on a pile in cohesive soils is probably one of the most difficult decisions that a foundation engineer has to make. This is because there are so many uncertainties involved. What should the penetration be? What is the magnitude of the skin friction? What will be the group effect? What will be the settlement under various conditions? What load can the underlying soils withstand safely? How can the greatest economy be secured consistent with safety? In a large job, too much conservatism can cost a lot of money; too optimistic a decision can result in a tremendous amount of dangerous or unsatisfactory construction.

The data given in the Foundation Code of the City of New York, quoted at the end of this article, apply to many features of pile foundations. They should be examined carefully, especially the provisions affecting the allowable loads on piles. Elsewhere will be found suggestions and recommendations somewhat at variance with this code. However, such variations should be expected. The code is prepared for the guidance of engineers in only one part of the United States, admittedly an important part. It should be studied carefully. Other people and other sections naturally have different ideas and customs.

Special attention is called to the provisions of paragraph $i(2)$ of the code in which the presumptive bearing value may be based upon the results of load tests of piles driven in advance at the site. This is a wise procedure and, in many cases, it may result in considerable economies when piles can be driven to firm materials and can be proved to have large load-carrying capacity. Such tests are far better than mere estimates. However, notice paragraph $i(4)$. This is very important and necessarily leaves in doubt the bearing value of piles in large groups when in or underlain by cohesive soils. At the expense of reiteration, the following warning is given:

Load tests of piles that are wholly or largely end-bearing piles supported upon firm soils will generally yield reliable results; those made on friction piles in and underlain by cohesive or weak soils will generally yield results that are considerably (or even radically) larger in value than the actual average safe bearing value per pile when used in a large group.

Table 10-1 gives a general scale of values that may be useful in determining the safe bearing value of piles and in comparing types. The data should be used for preliminary designs only. Tests should be made where such action is practicable. In small jobs, where small groups of piles are used in widely scattered locations or where soil conditions vary considerably, it may be unduly costly to make proper tests. Conservative bearing values should then be used in design, and these should be compared with such additional data as can be secured during the driving.

An interesting and instructive case showing an apparent change in the bearing value of piles was reported to the author by A. G. Beaulieu of Westcott and Mapes, Inc., New Haven, Conn. The story is approximately as follows:

An extension of a power plant along the Eastern seaboard was started before the war. The piles were driven, tested, and capped with concrete, then the work was stopped. They were Monotube piles driven down 40 or 45 ft. into sand. They were driven to practical refusal. After the war, when the extension was reauthorized, the equipment was quite different from that previously intended. As so often happens in such instances, the existing piles were now in the wrong places and they were not sufficiently numerous. The caps were to be knocked off some of them so that they could be incorporated in the new construction. Other piles were to be abandoned, and many new ones were to be driven.

The first few of the new piles were driven next to the existing structure. To everyone's surprise the new piles went down 8 to 12 ft. deeper than the old ones. Mr. Beaulieu ordered that one of the old piles be tested under load by jacking against girders fastened temporarily to four adjacent piles. This test pile settled considerably and easily, indicating that it should not be loaded as planned. The next step was to try to drive this pile after welding on an extension. It was driven down nearly 10 ft. farther. Redriving caused most of the other old piles to go down from a few to 13 ft. Load tests thereafter indicated that the redriven piles were safe for use as intended.

In trying to find the reasons for this, Mr. Beaulieu discovered that, when the original piles were being driven, the power company was experimenting with pumping its water supply from some deep wells that had been sunk on the property. It seems that the previous piles were driven when the watertable was lowered temporarily beneath them. The sand was then merely damp, and it was stiff because of this moisture. Of course, the piles were driven as far as they could be at the time. When the later piles were put in, the abandonment of the wells had allowed the water to return to within a few feet of the surface. The saturated sand then did not offer the same resistance that the moist sand had, and the piles could be driven much deeper. This seems to be a reasonable explanation of the phenom-

TABLE 10-1. Estimated Safe Bearing Values of Piles Used Singly or in Very Small Groups
(Loads are in tons per pile. All piles are 50 to 60 ft. long, laterally supported.)

Type of action	Soil	Type of pile									
		Wood	Precast concrete		Cast-in-place concrete, with shells				Steel pipe and concrete		H piles, 14 in.
			12 in. square	18 in. octagonal	Tapered	Cylindrical		Button bottom			
						12 in.	18 in.		12 in.	18 in.	
End-bearing	Firm, fine sand...........	20–25	20–25	25–30	20	20	25–30	20–30	20–25	25–30	
	Coarse sand............	20–25	25–30	30–40	20–25	20–25	25–35	20–35	20–25	25–35	
	Sand and gravel.........	20–25	30–35	35–40	20–25	20–25	35–40	30–40	25–30	35–40	
	Hardpan or caliche.......	.....	.....	40–50	.....	25	40–50	40–50	25–30	40–50	40–60
	Shale or disintegrate rock..	.....	.....	.....	.....	25	40–50	40–50	25–30	40–50	50–75
	Bedrock................	.....	.....	.....	.....	25	40–50	40–50	25–30	40–50	60–80
Friction	Compacted silt..........	15	12	.....	10						
	Soft clay..............	15–20	15–18	.....	15–20	15–20					
	Medium clay...........	20–25	20–25	20–25	20–25	20–25	20–25				
	Stiff clay..............	20–25	25–30	30–35	25–30	20–25	25–30				
	Fine sand (confined)......	20–25	20–25	25–30	25–30	20–25	25–30				

enon. The implications and importance of this may be very important in connection with future work where wellpoints are used to lower the water level temporarily, and where the watertable is subject to considerable variation.

It is unsatisfactory to leave such an important matter as the bearing value of an individual pile with only vague generalities as a guide. Yet it is difficult to do otherwise. Past experience is one of the best guides available in the making of a decision regarding the allowable load on an individual pile. Much should be done to assemble data in the future and to get them in such form that the depths, strata, and properties of soils can be compared with those of the site at which the proposed structure is to be built. Nevertheless, in Art. 10–17, the author hazards the presentation of some procedures and practical cases that may help to illustrate how to select the type of pile and the safe load for each.

The following information is quoted from the Foundation Code of the City of New York, adopted in 1948:[1]

Group 3

Pile Foundations

§ C26-405.0 **General requirements.**—*a.* Definition of a pile.—A "pile" is a structural unit introduced into the ground to transmit loads to lower strata or to alter the physical properties of the ground, and is of such shape, size and length that the supporting material immediately underlying the base of the unit cannot be manually inspected.

b. General.—All piles shall conform to the requirements of this Group 3 and of such other provisions of the Code as are referred to in Group 3.

c. Evaluation of supporting materials for pile foundations.—The bearing values of soils supporting pile foundations shall be evaluated by one of the following methods in accordance with the provisions of the sections specified herein; (*a*) the resistance to driving of piles, section C26-405.2, *h*; (*b*) pile load tests, section C26-405.2, *i*; (*c*) the resistance to jacking, section C26-405.2, *j*. The above values may be modified as required by section C26-405.2, paragraphs *e*, *f* or *g*. The presumptive bearing values contained in section C26-377.0 shall not apply to pile foundations.[2]

d. Protection of pile materials.—Where the boring records or site conditions indicate possible deleterious action on pile materials because of soil constituents or of changing water levels, such materials shall be adequately protected by approved preservatives or impervious encasements which will not be rendered ineffective by driving and which will prevent such deleterious action.

e. Wood piles.—

1. Wood piles shall be cedar, cypress, Douglas fir, hickory, Norway pine, oak, Southern pine, spruce, Western hemlock, or other similar species approved for such

[1] Courtesy of Councilman Hugh Quinn, head of the New York City Council's Committee on Buildings, who has been the leader in the revision of the Code.

[2] This information is given in Table 4-2.

use. Where required to be protected by preservatives, such treatment shall conform to the preservative treatment hereinafter specified.

2. All wood piles shall be of sound timber suitable for driving, cut above the ground swell, free from decay, unsound knots, knots in groups or clusters, windshakes and short or reversed bends. The maximum diameter of any sound knot shall be one-third the diameter of the pile section where the knot occurs, but not more than four inches in the lower half of pile length nor more than five inches otherwise. All knots shall be trimmed flush with the body of the pile and ends shall be squared with the axis. Such piles shall have reasonably uniform taper throughout their length and shall be so straight that a line joining the centers of point and butt shall not depart from the body of the pile. No bark or wane shall be measured in required dimensions. The diameter at any section is the average of the maximum and minimum dimensions at that section. All piles required to be treated shall be thoroughly peeled.

3. For temporary structures of a minor character as approved by the superintendent and for lightly loaded class 4 and class 5 structures, as defined in sections C26-242.0 and C26-243.0, located over submerged or marsh land, untreated wood piles having minimum diameters of four inches at the point and eight inches at the butt shall be permitted above high tide level provided the top five feet of each such pile remains exposed for visual inspection.

4. Wood piles not impregnated with an approved preservative shall not be used unless the cut-off or top level of the pile is below permanent water table level. The permanent water table level shall not be assumed higher than the invert level of any sewer, drain or subsurface structure, existing or planned, in the adjacent streets, nor higher than the water level at the site resulting from the lowest drawdown of wells or sumps.

5. Creosoted timber piles when pressure treated to a final net retention of not less than twelve pounds of creosote per cubic foot of wood may extend above permanent water level when installed and protected in accordance with the following provisions:

(*a*) The tops of the cut-off piles shall be below finished ground level and shall be treated with three coats of hot creosote oil and capped with concrete, equal to at least class A concrete as defined in section C26-365.0.

(*b*) The preservative shall be grade one coal-tar creosote oil as required by United States federal specification, No. TT-W-571-*b*. Preservative treatment shall be an empty-cell process, in accordance with the same specification.

f. Rolled structural steel piles.—Rolled structural steel piles shall conform as to material to the requirements of section C26-322.0. Sections of such piles shall be of H form, with flange projection not exceeding fourteen times the minimum thickness of metal in either web or flange and with total flange width at least eighty-five per cent of the depth of the section. No section shall have a thickness of metal less than three-eighths of an inch. Other structural sections or combinations of sections having flange widths and depths of not less than ten inches and thickness of metal not less than one-half inch may also be used.

g. Pre-cast concrete piles.—Pre-cast concrete piles shall be reinforced with longitudinal reinforcing equal to at least two per cent of the volume of the concrete in such piles and with lateral reinforcing in the form of hoops or spirals of at least one-quarter inch round rods or wires, spaced twelve inches on centers throughout the length of the pile, except in the bottom and top three feet, where this spacing shall be reduced to not more than three inches. The top of this pile may be cut off after driving. Reinforcing steel shall be covered with not less than two inches of concrete. All piles shall be properly cured before they are driven.

h. Cast-in-place concrete piles.—After installation to final depth and immediately before the placing of the concrete filling, the inside of the tube, shell or bore shall be free of any foreign matter. Concrete shall be placed by such methods that the entire volume of the tube, shell or bore is filled. Concrete filling shall not be placed through water, unless the superintendent specifically consents in writing to such placing, after the submission to him of the detailed method of procedure. The concrete cap shall not be poured until at least one hour after all piles within the cap group are completely filled.

i. Combination or composite piles.—Combination or composite piles may consist of two types of piles. The maximum allowable load shall be that allowed for the weaker section. The design of the piles shall be satisfactory to the superintendent. The connection or joint between the two sections shall be so constructed as to prevent the separation of the upper and lower sections during construction and thereafter. The details and methods of making joints shall be submitted to the superintendent and approved by him before any piles of this type are used.

j. Piles located in soils subject to physical change or movement.—

1. Structures on piles installed in unstable strata of soil which are or may be subject to lateral movements shall be adequately braced by batter piles or by other effective methods. All such piles, including the bracing piles, shall be driven to satisfactory resistance into material of class 11, or better as classified in section C26-377.0, *c*,[1] below the lowest layer of unstable material, or to rock.

2. Piles installed in soils which exhibit considerable subsidence and consolidation during driving, shall penetrate to satisfactory resistance into suitable underlying material or shall be driven to rock.

k. Use of existing piles at demolished structures.—

1. Piles left in place, where the structure has been demolished, shall not be used for the support of new construction unless satisfactory evidence can be produced as to the length and driving conditions of each pile, which evidence will prove that the piles in question are adequate for loadings in accordance with the requirements of this group 3.

2. Where additional piles are required to support the loadings of the new structure, then the existing piles shall be limited to seventy-five per cent of their rated load-carrying capacity as determined under subparagraph 1 above, and the additional piles shall be of similar type and shall also be restricted to seventy-five per cent of the rated load-carrying capacity as determined by the provisions of section C26-405.2.

l. Minimum overall pile dimensions.—Except as provided in section C26-405.0, *e*, 3, no tapered pile shall be less than six inches in diameter at any section, nor have less than an eight-inch diameter butt at cut-off. No pile of uniform section shall have a diameter of less than eight inches, or, if not circular, a minimum dimension of less than seven and one-half inches.

[1] See Art. 4-3.

Tapered shoes or points of lesser dimensions may be attached to the ends of piles.

m. Minimum spacing of piles.—Except as provided in subparagraph 4 below, the minimum spacing of piles shall be as follows:

1. Piles bearing on rock or penetrating into rock shall have a minimum spacing center to center of twice the average diameter or 1.75 times the diagonal of the pile, but not less than twenty-four inches.

2. All other piles shall have a minimum spacing center to center of twice the average diameter or 1.75 times the diagonal of the pile, but not less than thirty inches, except that all piles located in groups or abutting groups that receive their principal support in materials below class 6, as classified in section C26-377.0, *c*, shall have their spacing increased above the minimum values by ten per cent for each interior pile up to a maximum increase of spacing of forty per cent.

3. If, because of known obstructions or space limitations, piles are originally designed to be spaced closer than specified above, or if piles along a lot line are located less than one-half of the required spacing, from the lot line, the carrying capacity of each pile not sufficiently distant from another pile or from the lot line shall be reduced. The percentage reduction in load-carrying capacity of each pile shall be one-half of the percentage reduction in required spacing.

4. When the supporting capacity of a single row of piles is adequate for the wall of a structure, effective measures shall be taken to provide for eccentricity and lateral forces, or the piles shall be driven alternately in lines spaced at least one foot apart and located symmetrically under the center of gravity of the loads carried. A single row of piles without lateral bracing may be used for private dwellings not exceeding two stories in height, provided the centers of the piles are located within the width of the foundation wall.

n. Minimum penetration.—Piles shall penetrate into soil of class 12 or better, as classified in section C26-377.0, *c*, at least ten feet below cut-off level and at least ten feet below ground level. The pile point shall be at least ten feet below the nearest established curb level when the pile is located twenty-five feet or less from the lot or property line. Any embedment of such a pile in soil less than ten feet below the nearest established curb level shall not be considered as providing any resistance for such pile, and load-carrying determinations for such pile, in accordance with the provisions of section C26-405.2, shall be made after such embedment is eliminated, by casing off, by excavation, or by other acceptable means.

o. Bracing of piles.—

1. Tops of all piles shall be embedded in caps not less than three inches, and the caps shall extend at least four inches beyond the edge of all piles.

2. Except for single row piles permitted in section C26-405.0, *m*, 4, every pile shall be laterally braced by rigid connection to at least two other piles in radial directions not less than sixty degrees apart. Three or more piles, connected by a rigid cap, provided they are located in radial directions not less than sixty degrees apart, shall be considered as being braced.

3. Concrete ties for bracing piles shall have minimum dimensions of one-twentieth of the clear distance between pile caps, but not less than eight inches, and shall be reinforced as a column with the bars anchored in the caps to develop full tension value. A continuous reinforced stone or gravel concrete slab or mat six inches or more in thickness, supported by and anchored to the pile caps, or in which piles are

embedded at least three inches, may be used in lieu of ties for bracing if such slab does not depend upon the soil for the direct support of its own weight and any loads which may be carried thereon.

p. Soil under pile cap.—The soil immediately below the pile cap shall not be considered as carrying any vertical load.

q. Pile caps.—Pile caps shall be designed in accordance with the requirements of section C26-496.0 for the pile loads and butt dimensions, considering each pile as a separate reaction concentrated at the butt section.

§ C26-405.1 **Requirements for installation of piles.**—*a.* Precautions during installation.—Piles shall be installed with due consideration for safety of adjacent structures by a method which leaves their strength unimpaired and which develops and retains the required load-bearing resistance. If conditions which will cause serious deterioration of piles exist at the site, suitable measures to avoid such damage shall be employed. Special precautions shall be taken to protect from injury both the butt, and where deemed necessary by the superintendent, the tip of piles. If any pile is damaged during installation, the damage shall be satisfactorily repaired or the pile rejected.

b. Equipment.—Equipment and methods for installing piles shall be such that piles are installed in their proper position and alignment.

Followers shall be used only upon written permission of the superintendent and only where necessary to effect installation of piles. A follower shall be of steel of such size, shape, length and weight as to permit driving the pile in the desired location and to the required depth and resistance. Cushion blocks shall be of such materials and design that loss of energy is held to a suitable minimum.

c. Tolerances and modification of design due to field conditions.—If any pile is installed out of plumb more than two per cent of the pile length, the design of the foundation shall be modified as may be necessary to support the resulting vertical and lateral forces properly.

In types of piles which are impossible of subsurface inspection, a variance from the plumb of more than two per cent of the exposed section of the pile or other evidence which indicates that the piles are not installed within allowable tolerances shall be considered as sufficient cause for corrective measures.

Where piles are installed out of position and thus receive eccentric loading, the true loading on such piles shall be analytically determined from a survey showing the actual location of the piles as driven, and if the total load on any pile is more than one hundred and ten per cent of the allowable load bearing capacity, correction shall be made by installing additional piles or by other methods of load distribution.

Groups of piles shall not be modified by the addition of piles of lesser load values than the piles originally comprising the group.

A tolerance of three inches from the designed location shall be permitted in the installation of piles, without reduction in load capacity, provided the piles comply with the requirements of this subparagraph for conditions of eccentricity.

d. Jetting.—Jetting shall not be used except when permitted by the superintendent in writing. When jetting is used, it shall be carried out in such a manner that the carrying capacity of the piles already in place and safety of existing adjacent structures shall not be impaired. Jetting shall be stopped not less than three feet above the final expected pile-tip elevation and the piles shall be carried down at least three feet beyond the depth of jetting and until the required resistance is obtained. If there is evidence that jetting has disturbed the load-bearing capacities of previously installed piles, those piles which have been disturbed shall be restored to conditions meeting the requirements of this article by proper redriving or by other acceptable methods after the jetting operations in the area have been completed.

e. Piles installed without impact.—Piles may be installed by methods other than impact driving provided the bottoms of such piles bear on or in a material of class 9 or better, as classified in section C26-377.0, *c.*

f. Penetration measurements.—Penetration measurements for the purpose of determining resistance to driving shall not be made when pile heads are damaged to an extent which may affect measured penetration, nor immediately after fresh cushion blocks have been inserted under the striking part of the hammer and such measurements shall be made without interrupting the driving more than may be necessary for such measurements, except for necessary repairs, or for redriving heaved piles as provided in paragraph *j.*

Gross penetration per hammer blow is the downward axial movement of the pile as measured at an established point on the pile located not more than five feet above the ground surface.

Net penetration is the gross penetration less the rebound, or the net downward movement of the established point.

g. Pile settlement.—Gross settlement is the total amount of downward movement of a pile or pile group which occurs under an applied test load. Net settlement of a pile or pile group is the gross settlement minus the rebound which occurs after removal of the applied test load.

h. Resistance.—Resistance is defined as the number of hammer blows or the jacking pressure required to cause any definite net penetration.

i. Sequence of installation.—Individual piles and pile groups shall be installed in such sequence that the carrying capacity of previously installed piles is not reduced.

j. Heaved piles.—In soils in which the installation of piles causes previously installed piles to heave, accurate level marks shall be put on all piles immediately after installation and all heaved piles shall be reinstalled to the required resistance.

k. Splicing of piles.—Splices shall be avoided as far as practicable. Where used, splices shall be such that the resultant vertical and lateral loads at the splices are adequately transmitted. Splices shall be so constructed as to provide and maintain true alignment and position of the component parts of the pile during installation and subsequent thereto. Except for piles which can be visually inspected after driving, splices shall develop not less than fifty per cent of the value of the pile in bending. Proper consideration shall be given to the design of splices at sections of piles which may be subject to tension or to bending.

l. Inspection and control.—The owner shall maintain a competent licensed professional engineer, acceptable to the superintendent, on the site during pile installations to insure and certify that piles are installed in accordance with design and code requirements.

m. Identification of piles.—A plan showing clearly the designation of all piles by an identifying system shall be filed with the department before the installation of piling is started.

n. Records of pile driving.—A record shall be kept by the owner's representative of the total penetration of every pile and the behavior of such pile during driving. Any deviation from the designed location, alignment or load-carrying capacity of any pile shall be promptly reported to the engineer or architect of record and adequate corrective measures shall be taken. Plans showing such deviations and corrective measures shall be filed with the department. Upon the completion of the pile driving, all pile driving records, together with the records of such additional borings or other sub-surface information that were obtained during the installation of the piles shall also be filed with the department.

§ C26-405.2 **Allowable load on piles.**—*a.* General.—The foundation loads of structures on pile foundations shall be carried down to satisfactory bearing materials, so that the entire transmitted load is supported without causing damaging vertical or lateral movements. The pile groups of a foundation shall be proportioned as to relative size, as nearly as practicable, to produce uniform settlement and shall be designed to support the maximum combination of the following loads:

(1) All dead loads including the weight of the pile cap and any superimposed load thereon.

(2) The reduced live load specified in section C26-348.0.

(3) Lateral force and moment reactions, including the effect of eccentricity, if any, between the column load and the center of gravity of the pile group.

(4) That amount of the vertical, lateral and moment reactions resulting from wind loads in excess of one-third of the respective vertical, lateral and moment reactions computed from the dead and other live loads.

b. Allowable Axial and Lateral Loads on Vertical Piles.—The maximum load permitted on any vertical pile shall be the allowable axial load described herein applied concentrically in the direction of its axis. No lateral loads in excess of one thousand pounds per pile shall be permitted on a vertical pile, unless it has been demonstrated by tests that the pile will resist a lateral load of 200 per cent of the proposed working lateral load without lateral movement of more than one-half inch at the ground surface; and will resist the proposed working lateral load without a lateral movement of more than three-sixteenths of an inch at the ground level.

c. Allowable Axial and Lateral Loads on Batter Piles.—The resultant of all vertical loads and lateral forces, occurring simultaneously, in the direction of the axis of batter piles shall not produce stresses in excess of those established in this section. The remaining horizontal component shall not exceed one thousand pounds per pile unless it is demonstrated, as established in section C26-405.2, *b*, that such piles can safely resist greater lateral loads.

d. Structural Strength of Piles and Limiting Values of Stresses.—

1. Strength of Unbraced Piles: That portion of any pile which is free-standing in air or water shall be designed as a column considered to be fixed at a point five feet below the soil contact level in class 9 material or better, as classified in section C26-377.0, *c*, and ten feet below in any other material.

2. Handling and Installing of Piles: Piles shall demonstrate their capacity to be handled and installed to the desired total penetration and resistance, and to resist the forces caused by the installation of adjacent piles without structural injury.

3. Limiting Values of Stresses: The average compressive stress on any cross-section of a pile, produced by that portion of the design load which may be con-

sidered to be transmitted to that section, shall not exceed the allowable values listed below. As an alternative method for the purposes of this section, it may be assumed that for piles more than forty feet in length, installed in material of class 12 or better, as classified in section C26-377.0, *c*, seventy-five per cent of the load of an end bearing pile, as covered in section C26-405.2, paragraphs *e* and *f*, is carried by the tip. For friction piles, as covered in section C26-405.2, paragraph *g*, the full load shall be computed at the cross section located at two-thirds of the embedded length of the pile measured up from the tip.

(3.1) Timber piles: cedar, western hemlock, Norway pine, spruce or other woods of comparable strength—600 pounds per square inch; cypress, Douglas fir, hickory, oak, southern pine, or any woods of comparable strength—800 pounds per square inch. The maximum allowable load on a wood pile having a six inch point shall be twenty tons, and on a pile having a point of eight inches or more, the maximum allowable load shall be twenty-five tons.

(3.2) Concrete: Concrete for piles shall comply with section C26-38.0, and shall be controlled or average concrete. f_c is the allowable axial compressive strength, and f'_c is the twenty-eight-day compressive strength of the concrete, but f'_c shall not exceed four thousand pounds per square inch for computation purposes. The ratio, n, is defined as 30,000 divided by f'_c

For controlled concrete: $f_c = 0.25f'_c$.

For average concrete: Class A, $f_c = 500$ pounds per square inch and $n = 15$;
Class B, $f_c = 626$ pounds per square inch and $n = 12$.

The value n is to be applied only to reinforcing steel in precast concrete piles.

(3.3) Reinforcing steel: The steel unit stress, $f_s = nf_c$. Reinforcing steel in excess of four per cent of the average cross sectional area of the pile, and reinforcing steel in cast-in-place concrete piles except as provided in subparagraph (3.5) below, shall not be permitted any load-carrying capacity.

(3.4) Rolled structural steel piles and concrete-filled steel pipe, shells or tubes: Steel unit stress, $f_s = 9{,}000$ pounds per square inch, provided the pipe, shell or tube is at least one-eighth of an inch thick, and f_c shall be as provided for in subparagraph (3.2) above. Where injurious soil conditions exist, the steel shall be protected as provided for in section C26-405.0, *d*.

(3.5) Piles bearing on rock, consisting of a structural steel shape installed as a full length core, protected by a minimum of two inches of concrete, in a concrete-filled steel shell, at least as thick as No. 18 United States Standard Gauge which is to be left permanently in place.

The pile shall be formed by driving a casing containing a close fitting temporary core in such manner as to exclude foreign matter from the casing, or by driving an open ended casing which shall be cleaned to the bottom. The casing shall be driven to rock or hardpan overlying rock, to a final penetration of not less than eight blows to the inch of the last three inches, using a hammer which delivers a blow of at least twenty-two thousand foot pounds, either leaving the drive casing permanently in place or placing a light shell within it and withdrawing the drive casing; placing a structural steel shape within the casing or shell; filling the casing or shell with concrete, then immediately driving the H beam to refusal on rock before the concrete has set, as indicated by a rate of penetration of one-fourth of an inch or less under the last five blows, with the hammer striking a blow of twenty-two thousand foot pounds or more or equivalent. Then f_s for the core shall be 12,000 pounds per square inch, and f_c for the concrete shall be as provided in sub-paragraph (3.2) above, with no load value for the shell.

The load on such a pile shall not exceed 100 tons without tests, or 200 tons on the basis of tests as specified in paragraph *i*.

e. Piles Installed Open-Ended to Rock.—Concrete-filled steel pipe or shells installed open-ended to bearing on rock for the loads permitted in this paragraph shall have a minimum steel thickness of 0.3 inches. The piles shall be cleaned to the bottom and redriven or rejacked until the piles bear securely, without possibility of sliding, on class 1 or class 2 rock as classified in section C26–377.0, *c*. The allowable load on such piles, where satisfactory evidence is submitted that the piles are bearing on class 1 or class 2 rock, shall be determined by either of the following methods: (1) The load at the top of the pile shall not exceed eighty per cent of the load determined in accordance with the limiting stresses given in sub-paragraphs *d*-(3.4) and (3.5) for the combined steel and concrete section, provided that the pipe or shell shall be driven to resistance such that the net penetration for the last five blows totals one-quarter inch or less under the hammers specified in paragraph *h*, unless permission is granted in writing, by the superintendent, to permit the use of lighter hammers because of limited headroom due to existing overhead structures; (2) in accordance with the provisions of paragraph *i* of this section for loading tests if driven, and not more than fifty per cent of the jacking pressure, if jacked.

The maximum allowable load on any single pile of this type shall not exceed that permitted by the limitations for material stresses, soil conditions and other requirements of sections C26-405.0, C26-405.1 and C26-405.2, but in no case shall the allowable load exceed two hundred tons.

f. Piles Bearing on Rock, Hardpan or Gravel-Boulder Formations Directly Overlying Rock.—Except as provided in paragraph *e* of this section, the allowable load of piles bearing on rock, hardpan or gravel-boulder formations directly overlying rock shall be determined in accordance with paragraph *i* or by formula in accordance with the provisions of paragraph *h* for loads of forty tons or less per single pile or shall be determined in accordance with the provisions of paragraph *i* for loads exceeding forty tons per single pile, provided that in the latter case the piles bearing on rock are driven to resistance such that the net penetration for the last five blows totals one-quarter inch or less under the hammers specified in paragraph *h*, and piles bearing on hardpan or gravel-boulder formations directly overlying rock, are driven to resistance such that the net penetration for the last five blows indicates, in accordance with the formulas in paragraph *h*, a bearing value not less than the proposed pile value.

The maximum allowable load on any single pile of this type shall not exceed that permitted by the limitations for material stresses, soil conditions and other requirements of sections C26-405.0, C26-405.1 and C26-405.2, but in no case shall the allowable load exceed one hundred and twenty tons for piles bearing on rock, nor eighty tons for piles bearing on hardpan or gravel boulder formations directly overlying rock.

g. Piles Which Receive Their Principal Support Other Than by Direct Bearing as Covered in Paragraphs *e* and *f*.—The allowable load on piles which receive their principal support other than by direct bearing as covered in paragraphs *e* and *f* of this section, shall be determined in accordance with the provisions of paragraph *h* or *i* provided it is thirty tons or less per single pile; and for loads exceeding thirty tons per single pile in accordance with the provisions of paragraph *i* for load tests.

The maximum allowable load on any single pile of this type shall not exceed that permitted by the limitations for material stresses, soil conditions and other requirements of sections C26-405.0, C26-405.1 and C26-405.2, but in no case shall the allowable load exceed sixty tons.

Where the points of a proposed foundation are underlaid by a stratum of compressible soil ranking below class 10, as classified in section C26-377.0, *c*, either (*a*) the piles shall be driven completely through such compressible stratum to satisfactory bearing capacities in underlying material of class 9 or better, as classified in section C26-377.0, *c*, or (*b*) other effective measures shall be used to reduce the magnitude and unequal character of the settlement to be expected as a result of the consolidation of such stratum under the stresses imposed by the foundation loads, in which case a report shall be submitted by a qualified licensed professional soil engineer to the superintendent establishing the effectiveness of such measures, based upon laboratory soil tests on undisturbed samples of the compressible soils of a satisfactory quality and upon foundation analyses to determine to the satisfaction of the superintendent that the probable total magnitude, distribution and time-rate of settlement to be expected for the proposed structure will not be excessive.

h. Pile Loads Evaluated by Formula.—The following determination of the allowable pile load is to be used only where tests or experience have shown that formulas specified herein are applicable to the soil conditions shown by the borings and to the type of pile being considered. Where the existence of firm soil underlain by soil of poorer bearing value creates doubt as to the safe sustaining value of piles, or where for any other reason doubt exists as to the safe sustaining value of any pile, the superintendent may require that the site be investigated in accordance with the provisions of paragraph *i*.

The allowable loads may be determined by the value of R obtained by one of the following formulas, provided that the piles with an average diameter or side of eight inches or less are driven by a hammer which delivers a blow of at least seven thousand foot-pounds; that the piles with an average diameter or side greater than eight inches and not more than eighteen inches are driven by a hammer which delivers a blow of at least fifteen thousand foot-pounds; and that piles with an average diameter or side of more than eighteen inches are driven by a hammer which delivers a blow of at least twenty-two thousand foot-pounds. Double acting hammers shall be operated at full rated speed, pressure, and stroke as shown in the manufacturers' catalogues. The minimum hammer blow for piles intended to carry twenty-five tons or more shall be fifteen thousand foot-pounds.

For drop hammers: $$R = \frac{2WH}{s + 1}$$

For single-acting hammers: $$R = \frac{2WH}{s + 0.1}$$

For double-acting hammers: $$R = \frac{2E}{s + 0.1}$$

Where: R is the allowable pile load in pounds
W is the weight of striking part of hammer in pounds
H is the effective height of fall in feet
E is the actual energy delivered by hammer per blow in foot-pounds
s is the average net penetration in inches per blow for the last five blows after the pile has been driven to a depth where successive blows produce approximately equal net penetration.

i. Determination of Bearing Value by Load Tests.—When the allowable pile load is to be determined by load tests, the tests shall be made as provided below. Such load tests shall be made at the expense of the owner of the proposed structure, or of the person causing the piles to be installed. Before any load test is made the proposed apparatus and structure to be used in making the load test shall be approved by the superintendent. All load tests shall be made under the supervision of the superintendent or his representative. A complete record of such load tests shall be filed with the department.

(1) Uniform conditions: Areas of the foundation site within which the subsurface soil conditions are substantially similar in character, shall be established by borings not less than as required by section C26-376.0. Each such area shall be tested by driving at least three piles distributed over the area. Continuous records for the full depth of the penetration of the pile shall be kept of the blows per foot to drive the pile to the desired resistance. If the records of the driving resistance of these piles are not similar, or the driving resistance is not in reasonable agreement with the information obtained from the borings, or where piles designed to carry more than thirty tons each are to be installed in soils underlaid by soils of poorer bearing value, the superintendent may require additional piles to be driven for test purposes.

(2) Allowable pile load by load test: One of these three piles in each area of uniform conditions, but not less than two typical piles for the entire foundation installation of the building or group of buildings on the site, nor less than one pile for each fifteen thousand square feet of building area, shall be loaded by a method which will maintain constant load under increasing settlement. The test load shall be twice the proposed load value of the pile. The test load shall be applied in seven increments equal to one-half, three-fourths, one, one and one-fourth, one and one-half, one and three-fourths and two times the proposed working load. Readings of settlements and rebounds shall be referred to a constant elevation bench mark and shall be recorded to one one-thousandth of a foot for each increment or decrement of load. After the proposed working load has been applied and for each increment thereafter, the test load shall remain in place until there is no settlement in a two-hour period. The total test load shall remain in place until settlement does not exceed one-thousandth of a foot in forty-eight hours. The total load shall be removed in decrements not exceeding one-fourth of the total test load with intervals of not less than one hour. The rebound shall be recorded after each decrement is removed, and the final rebound shall be recorded twenty-four hours after the entire test load has been removed. The maximum allowable pile load shall be one-half that which causes a net settlement of not more than one-hundredth of an inch per ton of total test load or shall be one-half that which causes a gross settlement of one inch, whichever is less.

(3) Foundation piles: In the subsequent driving of the foundation piles for the structure, a pile shall be deemed to have a bearing value equal to that determined by the load test pile for that area of the foundation, when the foundation pile, using the same or equivalent make and model of pile hammer, and the same operation of the hammer with regard to speed, height of fall, stroke and pressure, and all other variable factors, shall develop equal or greater final resistance to driving than the load test pile. Where actual pile lengths vary more than fifty per cent from that of the test pile, the superintendent may require investigation to determine the adequacy of the piles.

(4) Pile groups: Where the superintendent has reason to doubt the safe load sustaining capacity of pile groups, he may require, at the expense of the owner, group load tests up to one hundred and fifty per cent of the proposed group load.

j. Piles installed by jacking or other methods without impact.—The carrying capacity of a pile installed by jacking or other methods without impact shall be not more than fifty per cent of the load or force used to install the pile. The carrying capacity of piles installed by static forces shall be demonstrated by load tests, on not less than two piles selected by the superintendent, applied over a period of time sufficient to indicate that excessive settlement will not occur. Carrying capacities shall not exceed the allowable loads as provided in section C26-405.2, *e*, *f* and *g*.

k. Underpinning piles.—Piles jacked into position for permanent and for temporary underpinning shall be evaluated for safe bearing capacity by the jacking pressures used. The working load of each temporary underpinning pile shall not exceed the total jacking pressures used to obtain the required penetration. The working load of each permanent underpinning pile shall not exceed two-thirds of the total jacking pressure used to obtain the required penetration if the load is held constant for ten hours, or one-half of the total jacking pressure at final penetration, but in no case are the load values set forth in section C26-405.2, *e*, *f* and *g*, to be exceeded.

10-4. Spacing of piles. When one plans the location of, or the pattern for, a number of piles in a group, there are several things that should be considered. Among them are the following:

1. End-bearing piles may be placed as close together as it is feasible to drive them—at least as far as the bearing capacity is concerned—if the stratum upon which they rest can support the loads.

2. Since friction piles are no better than the bearing capacity of the soil to which they transfer the loads, closely spaced piles may be inefficient and uneconomical.

3. It is usually inadvisable to attempt to drive piles closer together than twice the diameter of the butts, or a minimum of 2 ft. 6 in. A minimum of 3 ft. to 3 ft. 6 in. is preferable for cast-in-place concrete piles. Under certain conditions it may be possible to drive steel H-type piles and steel pipes more closely than stated herein, but this should be investigated carefully before it is planned.

4. Since it is often difficult to drive piles exactly in the position shown on the plans, the design should be so made that small variations in location

are not harmful. For example, assume that a small column load is to be supported by two piles, as pictured in Fig. 10-1(a). The column must be in the position shown. Now assume that the piles when driven are in the position given in (b). The eccentricity of the column with respect to the piles is very undesirable. It would have been better to plan upon a three-pile group in the first place, somewhat as illustrated in Sketch (c). Then, if the piles were not spotted accurately and if they were as indicated in (d), the result would not be too serious. Again, if the two-pile group were driven as shown in (e), the pile G would have to support almost the entire

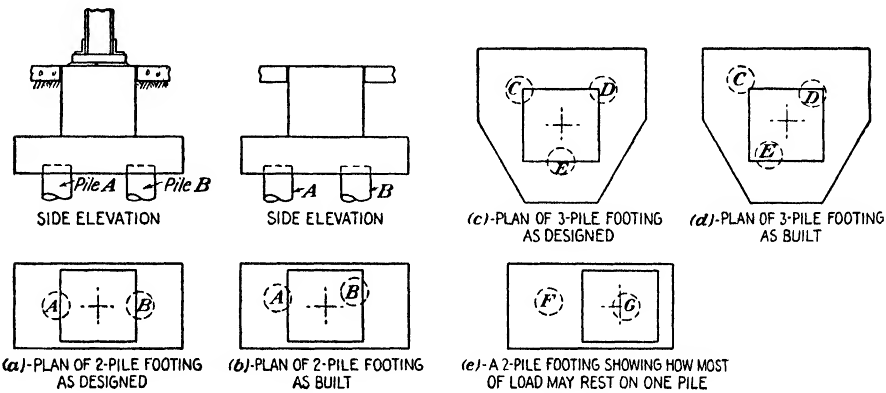


FIG. 10-1. Illustration of small footings on piles, showing possible effects of inaccurate driving of piles.

load because F could not offer much resistance without causing a tilting of the cap and probably a harmful bending of the column.

5. A reasonable spreading of the piles may be desirable to attain more adequate stability to resist overturning forces.

6. Close spacing of piles may reduce the size of the footings and the reinforcement needed in them. This may be important when a group of piles is large.

7. It is generally desirable to drive piles in "straight" rows rather than in staggered pattern when there are a great many in one group or area. However, this may not apply in the case of isolated footings such as shown in Figs. 10-2 and 10-3 where some sketches show the latter arrangement. Furthermore, in Fig. 10-4(a) it would be sufficient to use a single row of piles under the center of a light foundation wall except for possible tilting when the piles are not in the right location; a staggered pattern such as (b), or the scheme shown in (c), would be more stable. When there is a lateral overturning tendency, the wall might have the piles arranged as in Sketch (d), but some plan like (c) or (e) might be better.

8. When cast-in-place concrete piles are to be driven, it may be desirable to use a sufficiently large spacing to ensure that no harm will be done to adjacent empty shells or to concrete filling that has not cured adequately.

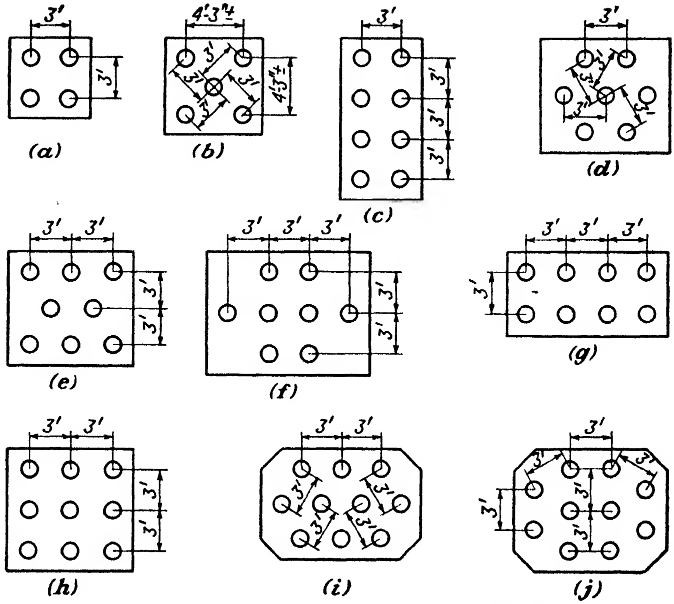


FIG. 10-2. Some patterns for isolated footings.

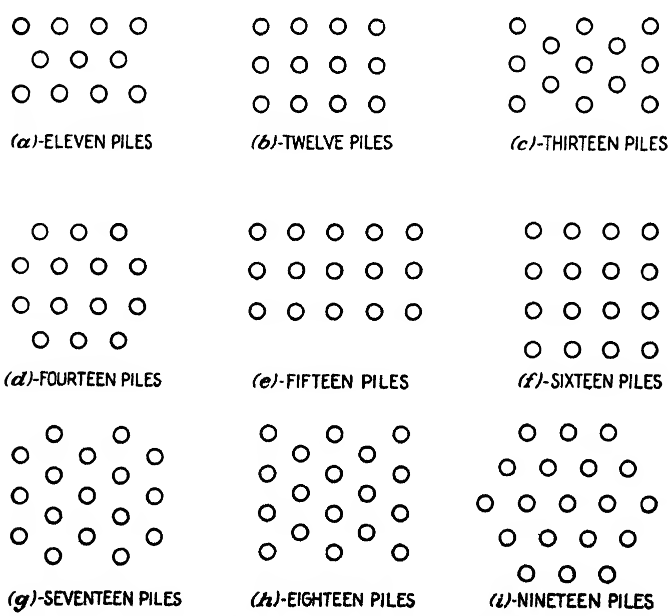


FIG. 10-3. Grouping arrangements for piles under large isolated footings.

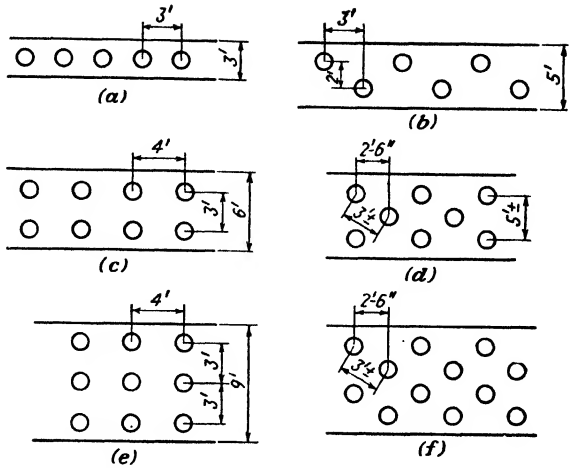


FIG. 10-4. Pile patterns for foundation walls.

10-5. Isolated footings on piles with vertical loads. Except when a single pile is sufficient, it is customary to use some sort of reinforced-concrete footing or cap to spread the concentrated load from a column over a group of piles. A stepped footing, pictured in Fig. 10-5, is used as a specific case to illustrate details and procedures. The pedestal is $4' \times 4'$.

Assume that this design is to be checked for the following conditions: $P = 475$ kips; rods a are nine ¾-in. rounds, and b are eight 1-in. squares; the maximum allowable superimposed load per pile is 25 tons; the piles are wooden friction piles approximately 35 ft. long; the dowels are twelve 1-in. rounds to match the column reinforcement; and all dimensions are as shown. The steps in the checking are as follows:

1. Load on piles. The footing weighs 56 kips (approx), including 1 ft. of earth on top. The load per pile is $(475 + 56)/12 = 44$ kips (approx), which is conservative but satisfactory.

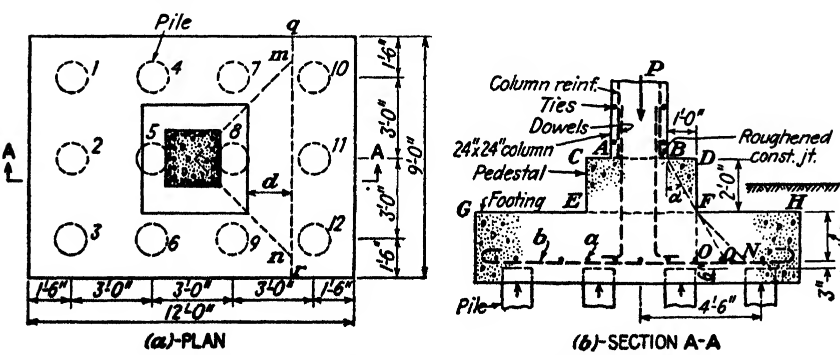


FIG. 10-5. A spread footing on piles with vertical loads.

2. Arrangement of piles. The 3-ft. spacing is reasonable. The diameter of the butt of a pile may be about 18 in. The edge distance of 18 in. should be adequate, and so is the embedment of 6 in. for the ends of the piles. The minimum cover of 3 in. over the reinforcement across the piles is desirable since they are wooden.

3. Pedestal and base of column. The pedestal appears to be satisfactory. The angle α is approximately 30°, as it should be. The arrangement of the dowels appears to be satisfactory because it matches the steel in the column.

4. Shear in footing. The gross load on each pile is 44 kips. Technically, the weight of the spread footing GH and the earth on it may be deducted from the total pile loads as far as the stresses in the footing are concerned. As an approximation, assume that the net load per pile is $(475 + 5)/12 = 40$ kips. The critical condition will be at E and F. The estimated average punching shear around the 4-ft. pedestal will be

$$v_T = \frac{V}{bkd} = \frac{10 \times 40{,}000}{(4 \times 48) \times 0.3 \times 21} = 330 \text{ p.s.i. } (< 0.2f'_c;\text{ safe})$$

As a maximum, assume that the footing at the edge F of the pedestal must resist the pile loads that are outside 45° lines from the center of the column, as shown in Fig. 10-5(a). Then,

$$v_T = \frac{V}{bkd} = \frac{3 \times 40{,}000}{48 \times 0.3 \times 21} = 397 \text{ p.s.i.}$$

Assuming that the loads are acting at the centers of the piles, 10, 11, and 12 are farther than 45° from F (2 ft. 6 in.). Therefore, the full net reactions on these piles will be assumed to cause longitudinal shear on the width mn of Sketch (a). Assuming 45° angles and $d = 21$ in.,

$$mn = 2(24 + 21) = 90 \text{ in.}$$

The diagonal tension (longitudinal shear) at this section outside of the 45° line FN in (b) may then be computed as

$$v_L = \frac{V}{bjd} = \frac{3 \times 40{,}000}{90 \times 0.9 \times 21} = 70 \text{ p.s.i. (safe)}$$

In this case could the full width qr of the footing be used for b in the above formula? Preferably not.

On the other hand, some specifications assume that, when piles are used, the diagonal tension shall be computed as though the critical section is that beyond FQ, where angle OFQ is 30°. In such a case, mn would equal approximately 71 in. and v_L would be at least 100 p.s.i., which is a bit too large. None of these assumptions should be considered as perfect; they are approximations at the best. The differences in computed values provide the engineer with some figures to use as a guide when he determines what dimensions to assign for the structure. In this case, the 21-in. depth seems to be none too large. Incidentally, if a footing is square, one may assume that one-half of the loads on the corner piles will be resisted by each of the adjacent trapezoidal sections when computing shears.

It is now evident that a designer might wish to place the point F, Sketch (*b*), so that the 45° line would hit the top of the outer piles by using a larger and deeper pedestal. However, since these piles may not be driven exactly as shown, since a lower v_L is desirable, since web reinforcement is impracticable, and since any possible weakness should be avoided, it seems to be advisable to increase d to 24 in. and to decrease the embedment of the pile ends to 3 in., keeping a 4-ft. square pedestal. The sides of the footing over piles 4 and 7, and 6 and 9, are obviously safe.

5. *Bending in footing.* Using the same general basis for bending about F of Fig. 10-5(*b*) as used for shear, piles 10, 11, and 12 will have a lever arm of 2.5 ft. from the edge F of the pedestal. Then

$$M_F = 3 \times 40{,}000 \times 2.5 = 300{,}000 \text{ ft.-lb.}$$

$$A_s = \frac{300{,}000}{18{,}000 \times 0.9 \times 2} = 9.2 \text{ in.}^2$$

$$\text{Required } \Sigma o = \frac{3 \times 40{,}000}{150 \times 0.9 \times 24} = 37 \text{ in.}^2$$

The reinforcement should then be about ten 1-in. square rods. In the other direction,

$$M_F = 2 \times 40{,}000 \times 1 = 80{,}000 \text{ ft.-lb.}$$

$$A_s = \frac{80{,}000}{18{,}000 \times 0.9 \times 2} = 2.5 \text{ in.}^2$$

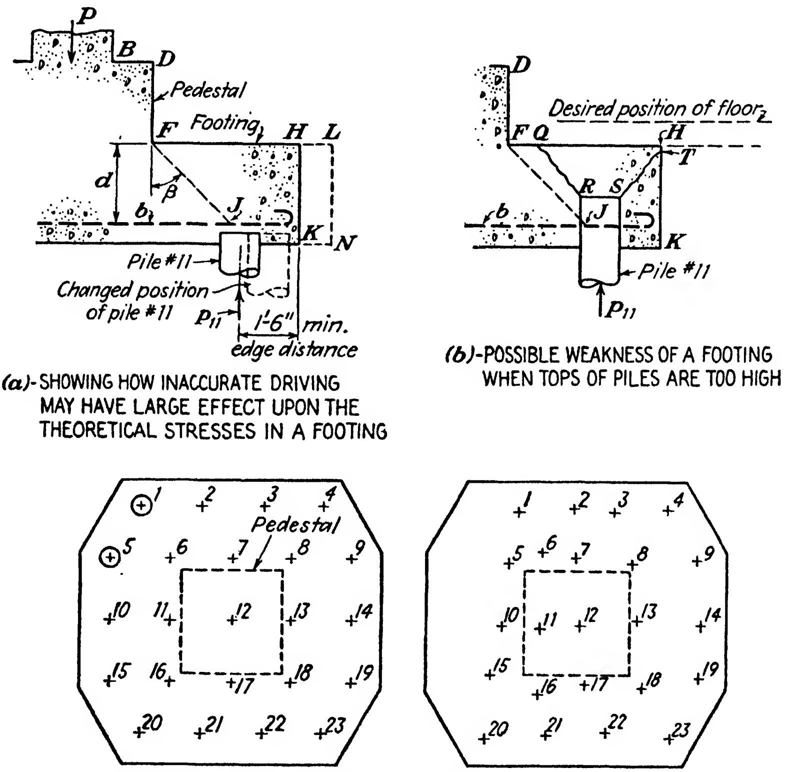
$$\Sigma o = \frac{2 \times 40{,}000}{150 \times 0.9 \times 24} = 25 \text{ in.}^2$$

The bond stress in the nine ¾-in. round rods will be excessive. Therefore, use thirteen ½-in. square rods for bars a in a band 8 ft. wide, with an extra bar about 1 ft. inside ends G and H.

In general, the preceding analysis shows that the position of the piles is very important when planning the footing because of the possible serious effect of the large concentrated pile loads upon the shear and bond in the concrete. Even though poured monolithically, the concrete below the tensile reinforcement should not be trusted on account of tensile cracking.

In further consideration of the question regarding whether or not a pile may cause serious diagonal tension in a footing, refer to Fig. 10-6(*a*). Assume that the design calls for pile 11 of Fig. 10-5(*a*) to be located so that, when β equals 45°, the line FJ intersects the steel above the top of the pile.

It might be argued that some of the reaction p_{11} will reach the pedestal without causing the part *FHKJ* to crack off. However, the pile may actually be driven as shown by the dotted lines. Then the argument no longer seems to have value. Unfortunately, the designer cannot be sure in advance just what the situation will be. Incorrect driving will affect



(a)-SHOWING HOW INACCURATE DRIVING MAY HAVE LARGE EFFECT UPON THE THEORETICAL STRESSES IN A FOOTING

(b)-POSSIBLE WEAKNESS OF A FOOTING WHEN TOPS OF PILES ARE TOO HIGH

(c)- PLANNED LOCATIONS OF PILES UNDER A LARGE FOOTING

(d)-POSITION OF PILES AS DRIVEN, SHOWING ECCENTRICITY OF LOAD

FIG. 10-6. Illustrations of how inaccurate driving of piles may endanger large footings.

bending as well as shear in the footing. If one provides strength enough to allow for moderate inaccuracies, he is conservative; if the pile is actually much closer to *F*, he is a bit wasteful. His opinion may not be asked before the piles are capped, and he should not risk important matters affecting life and property for insignificant economies.

It is desirable to make the contract drawings of important footings so that typical details illustrate what should be done if irregularities arise. For example, an adequate edge distance is desirable for the part of the footing beyond the outer piles. Perhaps the edge *HK* should be moved to *LN* in Fig. 10-6(*a*). Then the note calling for a minimum edge distance will control the construction. Furthermore, an increase of the leverage of the piles beyond F may affect the bending moment and reduce the safety factor more than is desirable. A note covering this might state that, if any of the piles are driven more than 1 ft. (or some other figure) farther from the center of the footing than shown on the plans, the chief engineer should be notified so that the footing may be redesigned.

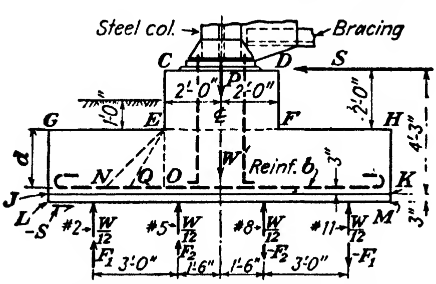
Another possible weakness is shown in Fig. 10-6(b). Assume that pile 11 is precast concrete, and that the top RS is 1 ft. too high. If rods b are placed so as to straddle the pile, will this be satisfactory? The principal danger comes from the tendency of the pile to punch out a section represented by $QRST$, and this may be very harmful. The pile should be cut off below rods b. If RS is 4 in. higher than intended in the design, should the pile be cut down, should the rods be raised 4 in. with the position of FH unchanged, should the whole footing be raised 4 in., should the rods be laid directly on RS with FH raised 1 in. higher? The answer is a problem for the engineer to settle in accordance with good engineering judgment. Of course, when the piles are below the intended elevations, they may be extended or the footing may be lowered or padded down to cover their tops.

Figure 10-6(c) pictures the design of a footing, whereas (d) shows approximately the pattern of the piles as actually driven because of a hidden obstruction or gravel pocket. Was this poor workmanship? It was done by experts who had considerable difficulty doing even this. The engineer is shown the result and is asked whether or not he approves proceeding with the footing which, incidentally, is to support a load of 1,400 kips. Is the effect of eccentricity dangerous? Yes. Should a combined footing be made and extended across to the next column at the left of the sketch so that a few intermediate piles can be driven? The latter seems to be reasonable. On the other hand, should the obstruction be blasted to pieces so that some piles can be driven down as intended? Probably this is too dangerous when the other piles have already been driven. At any rate, the eccentricity is too serious to be neglected. It seems best to try to cut through the obstruction by driving down a steel H, pipe, or mandrel in three places at the left side of the footing of Sketch (d). Then try to drive typical piles in these places after the steel has been withdrawn. If this is unsuccessful, the problem has to be solved in whatever practicable manner the engineer can invent.

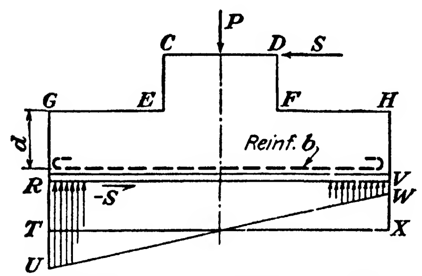
10-6. Isolated footings with vertical and horizontal loads. Consider again the 12-pile footing shown in Fig. 10-5(a). A longitudinal section is given in Fig. 10-7(a). Assume that the footing supports one leg of a braced steel bent which causes vertical and horizontal forces to be applied to the pedestal. The piles are surrounded by clay that has sufficient abutting power to prevent lateral tipping of the piles. The maximum bearing value of a pile for dead and live loads is assumed to be 25 tons, Inasmuch as wind forces are temporary loads, this allowable value will be increased 30 per cent when maximum vertical and wind loads are combined.

In order to illustrate a method for the computation of what will be called the load on each pile, and to analyze the footing, it will be assumed that $P_{LL+DL} = 475$ kips, $P_W = 60$ kips, $S = 40$ kips, rods b = ten 1-in.-square bars, and $d = 2$ ft.

1. Total forces. The weight of the footing and the soil on top of it will be included in the vertical loads acting on the piles. Therefore, $W = 475 + 60 + 55 = 590$ kips ($DL + LL +$ wind).



(a)-DIAGRAM OF CONCENTRATED PILE LOADS



(b)-SUBSTITUTE DISTRIBUTED PILE LOADS

FIG. 10-7. A spread footing on piles with vertical and horizontal loads.

2. Overturning. The wind shear S is assumed to have a lever arm of 4.25 ft., the distance of S above the tops of the piles if driven as intended. Then,

$$M = 40 \times 4.25 = 170 \text{ ft.-kips.}$$

3. Vertical loads on piles. It is assumed that the load on any pile is

$$p = \frac{W}{N} \pm \frac{Mc}{I}$$

where N = the number of piles under the footing, c = the distance from the center of gravity of the pile group to the pile in question, and I = the plane moment of inertia of the pile group about an axis through the center of gravity of the group and perpendicular to the plane of rotation. In this case, the axis is at the center line of the footing. Therefore, treating the piles as units that are concentrated at their longitudinal center lines, and neglecting the moment of inertia of each pile about its own center, I for the three rows is

$$I = 3 \times 2(1.5^2 + 4.5^2) = 135 \text{ pile-ft.}^2$$

Therefore, for pile 2,

$$p_2 = \frac{590}{12} + \frac{170 \times 4.5}{135} = 49.2 + 5.7 = 54.9 \text{ kips}$$

Similarly, $p_5 = 49.2 + 1.9 = 51.1$ kips, $p_8 = 47.3$ kips, and $p_{11} = 43.5$ kips. The maximum is less than $1.3 \times 50 = 65$ kips allowed.

These computations show that the footing is conservatively designed. It might be that one could use 11 piles arranged as in Fig. 10-3 with 3 ft. between rows and between the piles in each row. On this basis, using the same general dimensions for the footing, the pile loads would be computed as follows:

$$I = 2 \times 2(1.5^2 + 4.5^2) + 2 \times 3^2 = 108 \text{ pile-ft.}^2$$

$$p_{LL+DL} = \frac{475 + 55}{11} = 48.2 \text{ kips}$$

and

$$p_2 \text{ for } DL + LL + W = \frac{475 + 60 + 55}{11} + \frac{170 \times 4.5}{108} = 60.7 \text{ kips}$$

Therefore, this revised design is safe and slightly more economical. The bending and shear in the footing will also be reduced slightly. They may be estimated by computations similar to those explained previously.

It is possible, for the purpose of analyzing a pile footing, to treat the pile loads as distributed pressures similar to those that would be caused if the footing were supported upon soil. This substitute pressure diagram for the preceding problem is pictured in Fig. 10-7(*b*). However, the piles necessarily cause localized loads of large magnitude. It seems advisable to plan and analyze isolated footings of moderate size upon the basis of these concentrated loads. Large groups of piles under combined footings, mats, and large foundations may be replaced by assumed distributed loads when the designer believes that this is a more suitable and efficient procedure.

The stresses in the footing may be computed upon the same assumptions as those that were discussed in the preceding article. When the pile loads are computed, they may be used directly, or the average load per pile caused by the footing may be deducted, and the remainder used as the *net pile load*. Here again the angle OEQ in Fig. 10-7(*a*) is 30°; OEN, 45°. These are for consideration when computing the longitudinal shear. When the loads include dead, live, and wind forces, it may be permissible to increase the allowable unit stress in steel and concrete 30 per cent. The reader may test the footing for himself, to see if it is safe for this loading.

10-7. Isolated footings with eccentric loads. There may be cases in which a pile foundation is not or cannot be made so that the principal load coincides with the center of gravity of the group of piles. Of course, an engineer tries to avoid arrangements like this, but such a plan may be the most economical one to be devised under certain circumstances. If made properly, a footing with more or less permanent inequality of pile loads need not be feared. However, it is desirable to bear the following in mind:

1. End-bearing piles that rest upon a firm stratum may be loaded unequally without serious danger.

2. Friction piles that are in soft cohesive soils may eventually settle unequally if the loads on the piles in a group vary considerably and permanently.

3. The inequality of pile loads should be kept to a minimum.

4. Permanent downward pressure on the friction piles at one edge of a footing and uplift on those at the opposite edge should be avoided.

As an illustration, assume that Fig. 10-8(a) shows the location of a proposed column near the reentrant corner of an existing structure that is

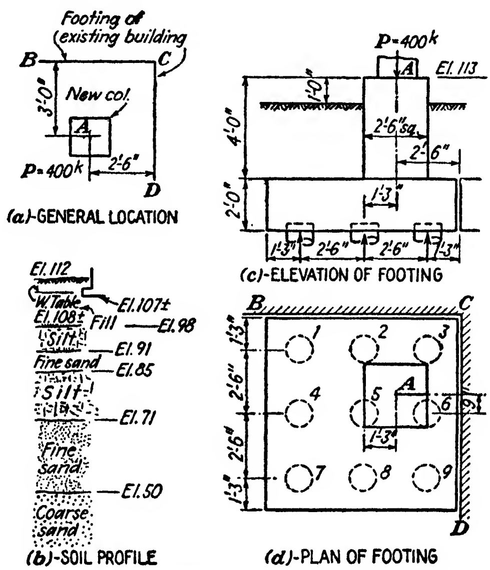


FIG. 10-8. An eccentrically loaded footing.

supported upon poured-in-place concrete piles 45 ft. long. The soil profile is shown in (b). Poured-in-place concrete piles with metallic shells are to be used in the new construction. If driven so that the tips are approximately at El. 58, each of these piles will be assumed to have an allowable bearing equal to 30 tons. The top of the pedestal is to be at El. 113. From Sketch (a), it is obvious that the column will be near one corner of a rectangular footing or pile cap.

To obtain an idea of the number of piles required, assume that the footing weighs 40 kips, and that the average load per pile is 50 kips. Then

$$N = \frac{400 + 40}{50} = 9 \text{ piles}$$

To minimize the eccentricity, assume that the piles are to be driven rather close together, as shown in Figs. 10-8(c) and (d). The trial footing in the former weighs 36 kips. Its weight is slightly eccentric, but this will be neglected.

Figure 10-9(a) shows the piles as assumed, and the point A where the column load of 400 kips will be assumed to be concentrated. The footing load of 36 kips is at J. The moments of inertia of the piles about axes EF and GH are the same, and

$$\begin{aligned} I &= 3 \times 2 \times 2.5^2 = 37.5 \text{ pile-ft.}^2 \\ M_{EF} &= 400 \times 1.25 = 500 \text{ ft.-kips} \\ M_{GH} &= 400 \times 0.75 = 300 \text{ ft.-kips} \end{aligned}$$

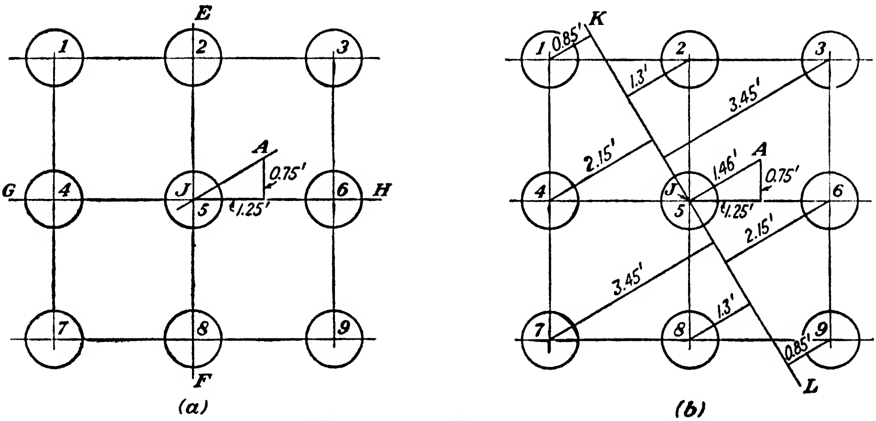


FIG. 10-9. Two methods of determining pile loads caused by eccentricity.

The load on pile 3 is the largest, and it may be estimated as follows:

$$p_3 = \frac{436}{9} + \frac{500 \times 2.5}{37.5} + \frac{300 \times 2.5}{37.5} = 48.5 + 33.3 + 20 = 101.8 \text{ kips}$$

This is obviously too large.

Another method of computation is illustrated in Fig. 10-9(b). The group of piles is drawn to scale in position. The line JA from the center of gravity of the group to the center of gravity of the column load is drawn, and its magnitude is scaled or computed. The axis KL is drawn through J perpendicular to JA. The perpendicular distances from the axis to the piles are scaled and recorded as shown. The moment of inertia of the group is the sum of the squares of these distances, *i.e.*,

$$\begin{aligned} I &= 2(0.85^2 + 1.3^2 + 2.15^2 + 3.45^2) = 37.9 \text{ pile-ft.}^2 \\ p_3 &= \frac{436}{9} + \frac{400 \times 1.46 \times 3.45}{37.9} = 48.5 + 53.2 = 101.7 \text{ kips} \end{aligned}$$

This result checks the preceding calculations, as it should. The method is shown only for the purpose of indicating its usefulness when a group of piles is very irregular, or when it turns out to be so, as actually driven.

It is now necessary to increase the number of piles, to change the design to a combined footing, or to devise some other means of supporting the structure safely. The problem should now be examined much more carefully. This will be done somewhat as an engineer might do when looking for and judging the merits of various solutions.

1. Increase the number of piles. To an experienced engineer, the arrangement of piles in Fig. 10-8(*d*) should automatically show a danger signal. When the resultant load is applied at the outer edge of the middle third of a rectangular footing on soil, the assumption of a uniform variation of pressure would cause the intensity at the edge of the footing on the "high" side to be twice the average unit pressure, whereas that on the other side would be zero. It seems reasonable to think of the action of a group of piles similarly. It is evident that point A of Sketch (*d*) is near the edge of the middle third of the group. Therefore, one should expect that the maximum load per pile would be somewhere near double the average load, and an assumption of 50 kips for the latter is obviously too large.

Using one-half the allowable load of 60 kips per pile, then, adding something for the weight of the large footing,

$$N = \frac{450}{30} = 15 \text{ piles}$$

If, in Fig. 10-8(*d*), six piles are added as pictured in Fig. 10-10(*a*), will this be satisfactory? By inspection alone, one can approximate closely the position of the axes EF and GH through the center of gravity of the group of piles. Although the figures are given, it is apparent that the time spent in computing them is wasted. At a glance, one can see that the load is far over to one corner of the group. The piles in the vicinity of A will take most of the load; those far away will resist uplift if they do any good at all. Just to show the reader the results of computations to determine the critical pile loads in this case, assuming that he does not believe the preceding statement, the following are recorded: $p_3 = 91$ kips downward; $p_{12} = 18$ kips uplift; $p_{13} = 18$ kips uplift. The extra piles are a waste of money, they do little to relieve the pressure on pile 3, and this plan is not satisfactory.

2. Driving the piles nearer to the existing footing. From Fig. 9-10, piles 50 ft. long may have a diameter of nearly 13½ in. at the butt. It is therefore unreasonable to expect to drive them closer than approximately 8 in. from BC and CD to the center of the pile. Furthermore, examine Fig. 10-10(*b*). Assume that KLM is the outside of the foundation wall and NOQ is the projection of the eaves overhead. To be investigated now are the clearances required for the pile-driving equipment. Can a pile driver snuggle alongside the building wall to drive such piles as 1, 2, 3, 6, and 9? Figure 10-11 will help to judge this. At best, one should not assume that the clearance RS of Fig. 10-10(*b*) can be less than 1½ to 2 ft. To reach pile 3, the equipment may approach the corner at an angle so that TU represents a line parallel to the leads. In general, TU should be at least 3 to 4 ft. For this situation and to encase the tops of the piles, it is probable that the piles should not be driven closer than 12 in. from the old footings. If they are placed 2 ft. from NOQ because of clearance needed for the equip-

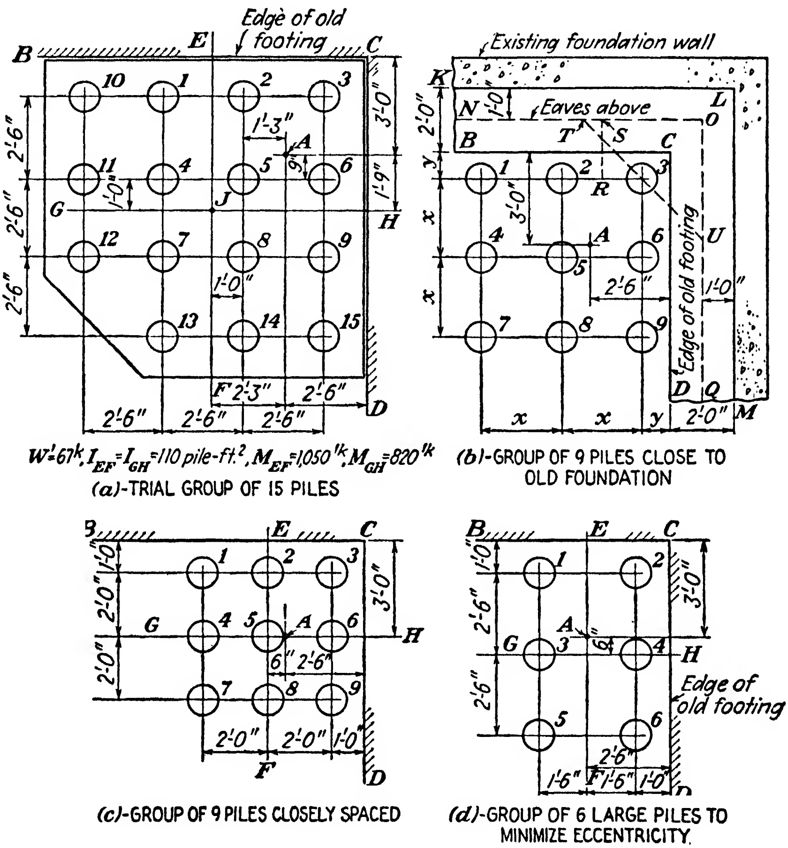
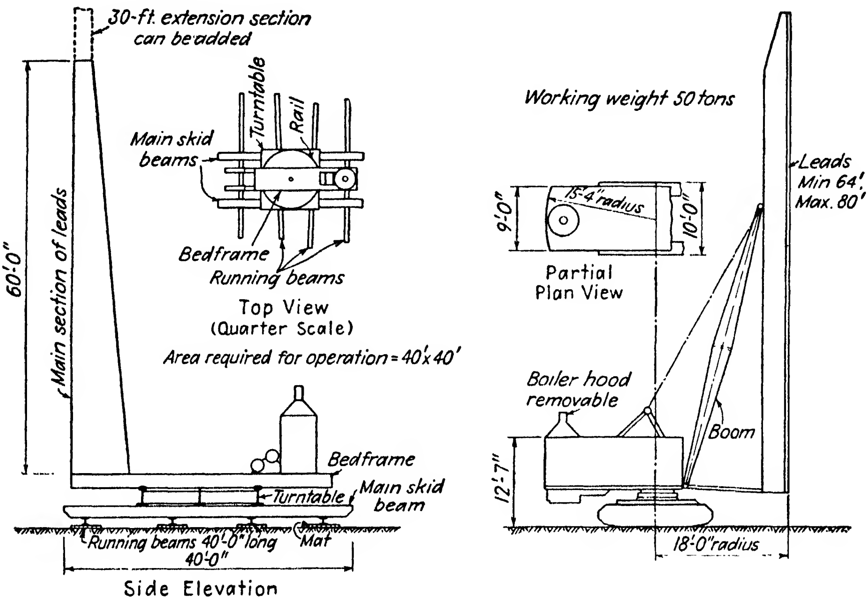


FIG. 10-10. A study of a pile foundation under eccentric loads.

ment and if the piles are 2 ft. 6 in. c.c., the effect of eccentricity will be remedied only slightly.

In such troublesome situations, one should remember that certain things called for on drawings may not be within the range of possibility. However, when he has been told subsequently that the piles have been driven as close as possible, yet they are 6 to 8 in. outside of the positions called for, he must accept them. He must now permit overloading of some of the piles or change the design of the structure. It is better to be sensible and conservative in the first place and not attempt to do the impracticable.

3. Drive the piles closer together. Suppose that the spacing x in Fig. 10-10(b) is reduced to 2 ft.; and y is 1 ft. The point A would then be 6 in. from pile 5, as shown in Sketch (c). Then $I_{EF} = I_{GH} = 24$ pile-ft.^{2}; $M_{EF} = 400 \times 0.5 = 200$ ft.-kips; and $p_3 = 65$ kips. Theoretically, this could be accepted, but it is practically impossible to guarantee that the



(a)-OUTLINE DIAGRAM OF A TURNTABLE SKID PILE DRIVER (Courtesy of Western Foundation Corp., New York, N.Y.)

(b)-OUTLINE DIAGRAM OF RAYMOND CRANE PILE DRIVER (Courtesy of Raymond Concrete Pile Co., New York, N.Y.)

General Data for Turntable Skid Pile Driver

	Wt., tons
1 two-drum hoist and 50 H.P. boiler	11
1 leads—60-ft. main section and 30-ft. extension	9
1 bedframe 36 ft. long	5
1 turntable 16 ft. × 16 ft. (with circular rail)	5
2 main skid beams 40 ft. long	4
	34
1 No. 0 Vulcan steam hammer	8
5 40-ft. running beams and 15 mats	10
Total shipping weight	52

Notes: Pile-driving equipment is very much specialized. Some drivers may have reach of 100 ft. and be able to drive 36-in. diameter piles. Others are made to drive short sections beneath 8- or 10-ft. clearances. Drivers are usually designed to handle only certain types of pile most efficiently. They may be made for railroad, floating, skid, or caterpillar supports.

Lengths of pile that may be driven if proper equipment is available (approx. limits):

	Length, ft.
Wooden	75
Precast concrete (not over 24"ϕ)	70
Composite wood and light-shelled cast-in-place concrete	125
Composite pipe and light-shelled cast-in-place concrete	200
Cast-in-place light-shelled concrete	110
Cast-in-place compressed concrete	60
Concrete-filled pipe (not over 24"ϕ)	200

FIG. 10-11. Some information regarding pile-driving equipment.

piles will actually be this way when driven. Experience shows that, even though the pile driver could do this, the lateral pressure caused by driving pile 5, for example, so close to 2, 3, and 6 is likely to cause the shells of these previously driven piles to collapse. If they are filled with "green" concrete, the concrete may be damaged. A spacing of 2 ft. 6 in. is close enough; 3 ft. is better.

4. Use a smaller number of stronger piles. The coarse sand below El. 50 of Fig. 10-8(*b*) is very firm. Assume that piles are to be driven down to El. 48. If so, will they support safely a load of 40 tons? The resistance to penetration into the coarse sand will be large. Jetting would assist in getting the piles down, but the action of the water might endanger the bearing value of the existing piles that now end at El. 62, 14 ft. above the desired position of the tips of the new piles. However, open-ended pipe piles might cut into the sand sufficiently without great difficulty. Steel H piles might also be driven into the coarse sand satisfactorily, but corrosion may be a question if ground water is flowing through the fill, silt, and sand strata. Precast concrete piles probably could not be driven without jetting. Poured-in-place piles with shells and small tips might not have sufficient end area or strength as columns. Pedestal piles would be difficult to make in a closely spaced group when the deep fine sand or coarse sand must be displaced. It seems to be advisable to use steel pipe piles with open ends and with the pipes left in place for protection of the newly placed concrete. However, if these are different from the piles used for most of the job and if there are only a few of them, it may be desirable to use the same type throughout. An analysis of the group shown in Fig. 10-10(*d*) indicates that the probable maximum load on piles 1 and 2 will be at least 90 kips each, which is still too large. Before this is accepted, one such pile in the vicinity should be subjected to a load test.

5. Use a small open caisson. It may be possible to sink a small pipe caisson under the center of the column. Such construction is discussed in Art. 12-5. However, if there is only one or a few of these to be used, the cost of securing the necessary equipment may be prohibitive.

6. Use a combined footing. If another column is along wall *BC* or *CD* of Fig. 10-8(*a*) and if it is near enough, a combined footing might be used. This type of construction is illustrated more fully in Art. 10-8. Probably this construction is the most practicable.

7. Revise design of superstructure. Sometimes the most practicable solution for troubles of this sort, especially when loads are large, is one of the following:

a. Set the new structure far enough from the old one so that reasonable footings can be built easily and economically.

b. Offset the columns in the lower portion of the structure so that the foundation condition is satisfactory. Then support the upper portion of the columns on girders cantilevered across these stub columns *c*. Set all columns as desired for the foundations, and cantilever the floor and wall framing past these columns.

8. Conclusion. The owner should be shown the advantages of action in accordance with item *7a*. If this is impossible, the most practicable solution is likely to be the use of a combined footing. This problem demonstrates the desirability of planning foundations and of making trial calculations in the early stages of a project instead of neglecting such matters until the design of the superstructure and equipment has progressed so far that changes will not be accepted, at least not without an argument.

10-8. Combined footings on piles. There may be situations in which it is desirable to use combined footings with piles under them. As a specific

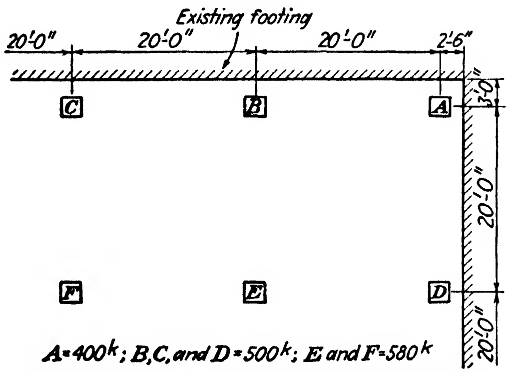


FIG. 10-12. Columns, locations, and loads for a new building adjacent to an existing one.

illustration of the planning of such construction, assume conditions similar to those shown previously in Figs. 10-8(*a*) and (*b*). Figure 10-12 pictures the corner column A and some of the neighboring columns of a large addition to be built alongside an existing building. Of course, the piles under the present structure are inadequate to support the new one if new loads are added to the present loads. Supposedly there are good reasons why it is desirable to locate the new columns in the positions shown. The column loads given in Fig. 10-12 do not include the weight of the substructure. Again assume that the new piles are to be designed for a bearing value of 60 kips each.

By adding an assumed weight of 50 kips to each of the column loads to allow for the weight of the substructure, the trial number of piles needed under each column is the following: at A, $450/60 = 8$; at B, C, and D, $550/60 = 9$; at E and F, $630/60 = 11$.

What is the best arrangement for the piles? As usual, different schemes are possible. One is shown in Fig. 10-13(*a*). This utilizes walls along the outer column rows, and the piles are arranged in two rows under narrow footings so that the typical construction is shown in (*b*). The piles under A

are grouped so as to be as close to the column as possible, and to have those that are off center so placed that they cause bending but no twisting in walls AB and AD. Thus at B, C, and D, the piles are arranged so that the substructure constitutes a sort of elongated footing that is symmetrical about the center of the column. The footing between the pile groups might be notched back as shown by the dotted lines in Fig. 10-13(a) if the

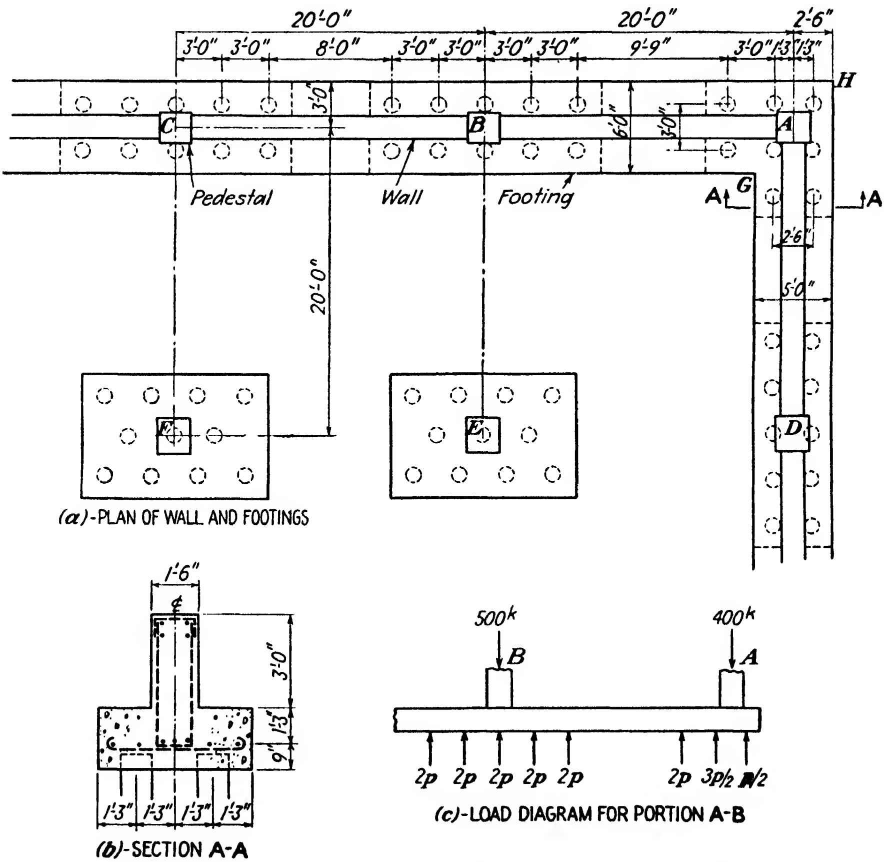


FIG. 10-13. Illustration of a continuous, combined, exterior footing.

bending strength of the wall will permit it, but this economy is not great. Of course, the piles could be spaced equally along the wall, but this would increase the bending moment in the latter and would accomplish no useful purpose here. If the wall were the outside of a deep basement, it would be so strong anyway that the effect of bending would be unimportant even when the piles were equally spaced.

In the computations for bending moments, shears, and bond in the walls, it is generally satisfactory to assume that the dead load of such a section of substructure as AB of Fig. 10-13(a) is distributed uniformly. Then the net pile loads may be used in the calculations. In this case, it is sufficient to average the loads on the piles under A and B, then use this figure as the load p shown in the load diagram in Sketch (c). Here the corner loads under A are assumed to be divided between members AB and AD as though the members were cut along the diagonal GH of Sketch (a).

Another variation is pictured in Fig. 10-14(a). This shows a long narrow thick footing extending from A past B. The piles between these points are distributed equally and so that the resultant of the loads on A and B almost coincides with the center of gravity of the pile group. At first glance this might seem a reasonable design. However, one must guard against surprises when planning heavily loaded members like these. An approximate check of this footing should be made before the scheme is adopted. This may be done with sufficient accuracy as follows:

$$\text{Average pile load} = \frac{400 + 500}{18} = 50 \text{ kips}$$

$$\text{Substitute uniform load} = \frac{2 \times 50}{3} = 33 \text{ kips per ft.}$$

$$\text{Approximate } M \text{ at center} = \frac{33 \times 20^2}{8} = 1{,}650 \text{ ft.-kips}$$

$$\text{Approximate } A_s = \frac{1{,}650}{18 \times 0.9 \times 2.75} = 37 \text{ in.}^2$$

Approximate v_L at 45° line from bottom edge of pedestal, using the uniform load, equals

$$\frac{33{,}000(10 - 1.25 - 2.5)}{72 \times 0.9 \times 28} = 113 \text{ p.s.i.}$$

Both A_s and v_L show that this footing is too shallow for good design, and the computations need be carried no further. It is therefore desirable to increase the depth, to use a T section as in Fig. 10-13(b), or to shift the 10 piles for B farther to the left so as to develop more negative bending under B and thus relieve the moment and some of the shear in AB. A combination of these ideas is preferable.

Elongated footings parallel to the outer walls might be used for columns C, D, etc., of Fig. 10-12. On the other hand, an arrangement like DE of Fig. 10-14(a) may be adopted with section B-B similar to A-A if conditions make this desirable. Here the portion under E is planned primarily as an isolated footing to carry the load at E, then the few piles between D and E cause bending in the portion DE. In other words, the elongated footing is joined to E in order to anchor that end and to eliminate the twisting that would occur otherwise.

Figure 10-14(*c*) is prepared to show one scheme that might be used if there were some obstruction between *A* and *B* and between *A* and *D* so that no continuous wall or footing could be used between them. The footing at *A* is planned first as though it were an isolated one with eccentricity. That at *E* is planned as a symmetrical isolated footing with its long axis parallel

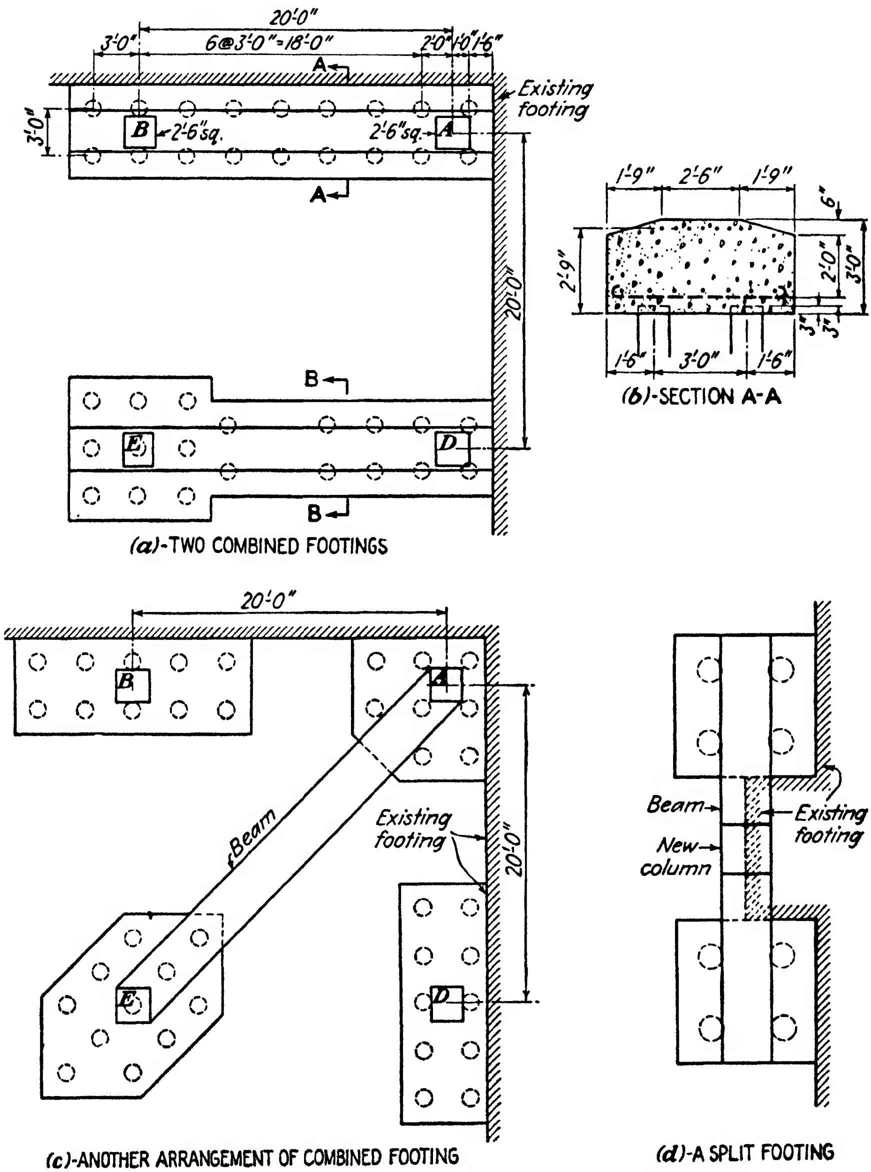


FIG. 10-14. Some other arrangements of combined footings.

to AE. The beam between them is then designed to resist the bending caused by the unsymmetrical loading at A.

It often happens that the piles under an existing foundation are grouped under a footing that projects, as pictured in Fig. 10-14(d). Sometimes they can be grouped at each side in this manner with concrete or encased steel beams to carry the column load across to the caps.

In such problems, an engineer has to study the particular case and then invent an arrangement that will serve his purpose most advantageously. The illustrations here are merely for the purpose of showing possible ways of attacking such problems.

10-9. Mats on piles. In many respects, the design of a heavy reinforced-concrete mat on piles is similar to its design if it were supported directly on soil. Except for local shearing stresses and punching action, piles that are spaced uniformly under the mat may generally be replaced for purposes of computation by an equivalent uniformly distributed load. For example, assume that the wooden piles under a mat are to be driven in rows 4 ft. on centers both ways, and that the net load on them exclusive of the mat is 18 tons each. This is equivalent then to an average upward pressure of

$$\frac{18 \times 2}{16} \quad 2.25 \text{ k.s.f.}$$

However, the bending moments in a mat so loaded and having columns 20 ft. c.c. are surprisingly large. One must be sure, too, that the concentrations can be spread safely and that the mat is stiff enough to avoid overloading of the piles near the columns because of local sagging of the mat at and near each column.

It is generally desirable to group the piles more closely under the portion of the mat that is near and beneath the columns, just as though isolated footings were used. The mat may be needed as a floor for a basement, or even as a first floor. It may then span from pile group to pile group somewhat as though it were a flat-slab floor on column capitals, or a few widely spaced piles may be placed under it to relieve the bending when the live loads are heavy. However, one should be careful how he does this.

Figure 10-15(a) shows a typical portion of a thick mat that was used under a power plant. The left-hand half of the picture shows the details originally proposed. The poured-in-place 30-ton piles under the thickened portion were designed to support the column loads of 1,000 to 1,500 kips, just as though there were no mat. Then additional piles were driven about 4 to 5 ft. c.c. in the intervening spaces to support pump foundations, a 5-ft. backfill above the mat on which the first floor was to be placed, and the heavy loads on the latter floor. These intermediate piles were assumed to support no more than 10 to 12 tons each, but they were the same as the others and were driven to the same depths.

Now try to visualize what might happen when the final loads are applied to the substructure and the 50-ft. friction piles are fully loaded. When the mat is poured, all piles will have small and almost equal loads. As the column loads are applied, the piles under the "footing" will settle slightly. The 3-ft. mat will try to spread these loads over the intermediate piles also. If it is not strong enough to do this, it will crack around the footing. If the mat is strong, it will distribute the loads about equally to all the piles. Thus the average load per pile may be 20 to 25 tons, and the intermediate

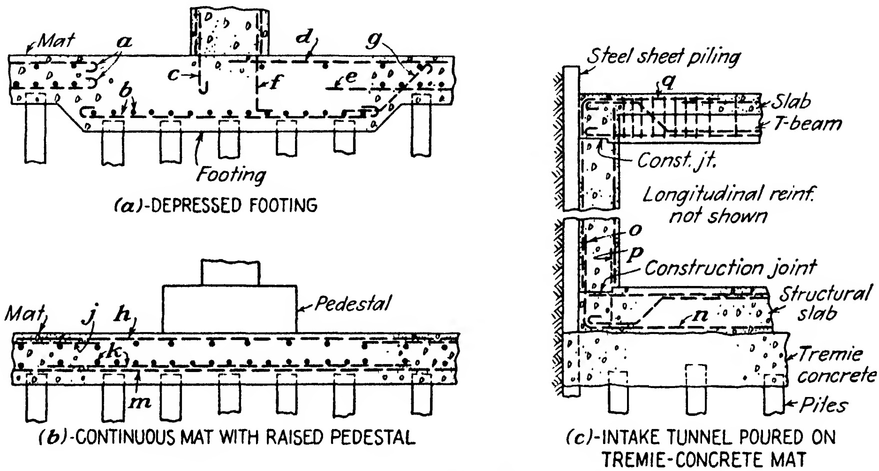


FIG. 10-15. Some arrangements of reinforced-concrete mats on piles.

ones will cause severe bending and shear in the mat. This is another one of those cases in which the structure should be considered as an entity instead of an assemblage of independent parts.

Refer again to the details shown in Fig. 10-15(*a*). It is obvious that, if the reinforcement is as detailed originally and as shown on the left-hand side of the drawing, the mat can crack easily between the ends of rods *a* and *b*. None of these can develop the required tension at the bottom of the mat where it joins the footing. The rods *b* will probably be overloaded because the upward shear from the mat will be applied at the edge of the footing where it will have a maximum lever arm. The dowels *c* are not detailed properly either.

The revised design is shown in the right-hand half of Fig. 10-15(*a*). The piles being already driven, it was necessary to use them. Rods *b* were therefore increased. The hooks on *e* were straightened out and the rods made heavier. Bars *d* were straightened also and some of them extended as shown. Then rods *g* were added around the edges to serve as stirrups and as a sort of hanger to attach the mat to the footing. The dowels *f* were made so that they could be supported on *b* during construction and so that the inadvisable use of a hook to resist compression could be avoided.

When it is possible to do so, the construction shown in Fig. 10-15(*b*) should be used instead of that in (*a*) for the following reasons:

1. The load from the column can be distributed more directly into the mat by shear and compression. It is not advisable to have to use suspenders like *g* in Sketch (*a*) if they can be avoided.

2. The tension in the bottom of the mat can be taken by full-length band rods *m*, with extra rods *k* under the column region. This avoids carrying heavy tensions around corners, as with rods *e*, *b*, and *g* of Sketch (*a*).

3. Rods *j* and *h* can be used as bands to resist tension in the top of the mat—the intermediate areas.

4. The pedestal can be made of whatever area and depth are necessary to prevent excessive punching shear and diagonal tension in the mat.

5. The tops of the piles can be cut off at the same elevation so that no depressions and special forms are required. Of course, as a general principle and as stated previously, it is desirable to group the piles under the mat so that they are near the column points, thus reducing the bending in the mat itself. The point emphasized here is that the piles at intermediate locations will resist loads if they can. This fact should not be forgotten.

Another example of the use of a mat is shown in Fig. 10-15(*c*). This illustrates the construction used at the intake to a power plant at tidewater. Heavy sheet piling was driven around the area required for the structure, the fill and muck were excavated considerably below the invert, and the wooden piles were driven under water by means of a follower. A tremie-concrete mat some 3 or 4 ft. thick was then placed below the future invert, and its top was roughly screeded to grade. When this had hardened sufficiently, the cofferdam was unwatered safely. The weight of this pad did not theoretically counteract the 18-ft. head of water, but the resistance of the piles and sheet piling to upward pull probably took care of the difference.

The structural slab of the invert consisted of a heavy mat that extended under the outer walls and under a heavy middle longitudinal wall. When the concrete substructure was completed and the superstructure was built, the structural mat acted as a one-way two-span slab to spread the loads from the walls to the piles through the now inactive tremie concrete. When the gates were closed and one-half of the twin structure was unwatered, the empty side resisted the upward and lateral hydrostatic pressures as a box. The structure was not waterproofed because minor seepage would not be troublesome.

Another case of what seemed to be a mat was really not a mat in this sense. A one-story warehouse with a large floor area was to be built for a grocery concern on a former low river flat having a thin blanket of new fill over 20 ft. or more of silt underlain by very fine sand and then by medium sand. The structure was planned with light columns approximately 28 by 54 ft. c.c. to hold the roof. The reinforced-concrete floor was designed as a flat slab. Single 50-ft. poured-in-place concrete-and-shell piles were driven in rows 14 ft. apart and 18 ft. on centers in each row except under the columns, where two piles were used. The single piles were capped with 3-ft.-square capitals and 6-ft.-square drop panels, whereas similar rectangular ones were used under the intermediate building columns. Continuous grade beams were built under the outer columns. All were poured monolithically with the floor slab. Each pile or pair of piles served practically as a column, whereas the "mat" was in effect nothing but a self-supporting floor.

The situation may be very different for pier platforms, relieving-platform types of bulkheads, large one-story warehouses, industrial plants, and other structures that have widely spaced columns with light superstructures but must support large trucks, trains, localized heaps of heavy goods, and machines. In these cases, the piles are necessarily located so as to support directly heavy fixed loads such as tracks and machines. Under extensive floor areas, the piles should be driven in rather widely spaced rows or in whatever arrangement suits the conditions so that there are sufficient piles to hold up a large load in any possible location without overstressing the mat or floor. It is advisable to make such a mat thick and stiff, with reinforcement in both directions at the top and bottom. Two of the principal weaknesses are likely to be the punching shear just above the top of the perimeter of the pile and the diagonal tension in the concrete at the surface of an imaginary 45° frustum of a cone sloping upward and outward from the edge of the top of the pile. The latter probably is the more critical. It is usually desirable to make the mat above the piles thick enough to avoid the need for web reinforcement.

When a mat is sufficiently thick, it may be entirely satisfactory to place the bottom reinforcement between the piles and below their tops. This is because the compression in the concrete is seldom critical in such a case, and even wooden piles will do no real harm to its resistance. However, the greater depth to the plane of the reinforcement will decrease the amount of reinforcement required.

10-10. Differential settlements. Besides those differential settlements that may occur because of differences in applied loads and in the qualities of portions of the substrata, sometimes there are differences in the piles themselves and in their action. For example, Fig. 10-16 shows the case of some wooden piles driven in 1931 for an extension of a power plant. Because of the business situation the work was discontinued. When the extension finally went ahead in 1946, recent developments in water-jacketed boilers had been so great that the new structure was to be vastly different from that originally contemplated—seemingly a characteristic of one's plans for future extensions. The new loads were so much heavier than the original ones that Raymond concrete piles were decided upon.

When planning the new work, the engineers were faced with the problem of deciding what to do with the existing 190 wooden piles. Should they be used, discarded, or pulled out? Since it was possible to make the layout so that the new screen house and intake could be placed upon the wooden

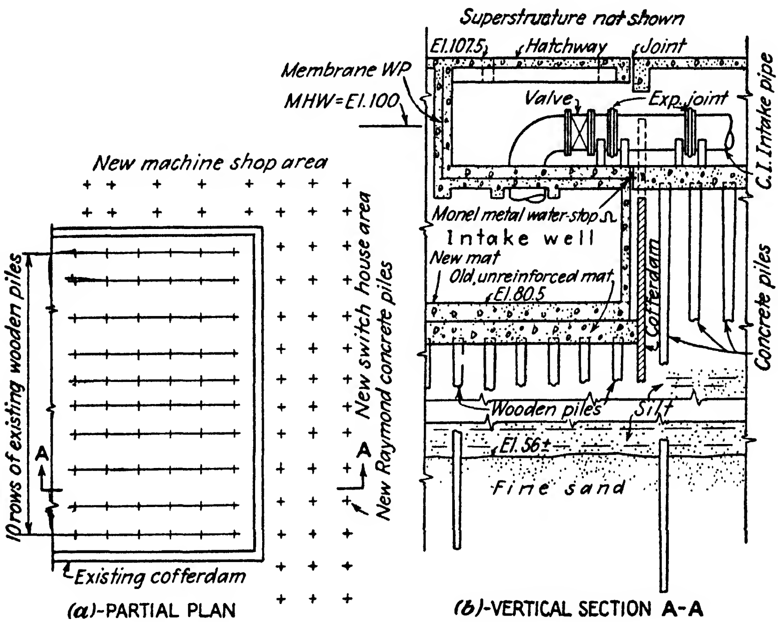
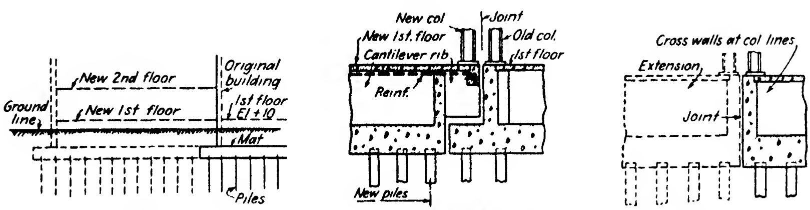


FIG. 10-16. Pile foundation for part of a power plant.

piles, the engineers decided to use them. The adjoining switch house was to be founded upon new Raymond piles. From the standpoint of the superstructure, it was desirable to make the screen house an integral part of the switch house. However, would the old concrete-capped wooden piles and the new concrete ones compress and settle equally? If they did not, the concrete foundations and the brick walls would inevitably crack. Since the wooden piles were driven for loads of 15 tons each, whereas the new piles were to support 25 tons apiece, equal movement at the tops seemed to be improbable, and reinforcing the concrete to prevent cracking was practically impossible; at least, the results would be uncertain and cracking would be difficult to remedy. Therefore, the two structures were isolated, as shown in Fig. 10-16(*b*), in order that vertical movement could occur without harm to the structure. Two expansion joints were used at the ends of the large cast-iron intake pipe that passed over the joint so that this piece could rotate slightly in a vertical plane; otherwise, settlement might crack the piping and cause a shutdown of the plant.

Another problem that faces one when making extensions to existing piled structures is illustrated in Fig. 10-17(*a*). This pictures an existing plant that is shown by the solid lines; the proposed extension is indicated by the dotted ones. The original footings and piling projected beyond the end wall in order to avoid or reduce the eccentricity of loading produced by the end columns and the heavy brick walls. The removal of the latter will reduce loads on the piles somewhat, but the principal rebound of the foundations will be only that of the elastic recovery of the piles and ground. The new piles to be driven alongside will be loaded gradually as the addition is built. Compression of the new piles may not be large, but compaction of the soil supporting them may be appreciable, and it will undoubtedly affect the neighboring soil under the adjoining end of the old structure, especially if the subsoil is plastic.



(*a*)-POSITION OF EXTENSION (*b*)-EXTENSION ISOLATED FROM ORIGINAL BUILDING (*c*)-OMISSION OF PROJECTING MAT

FIG. 10-17. Extension of an existing building when on piles.

In such a case, at what elevation should the floors in the new structure be built compared to those in the existing plant? If they are placed at the same elevation and if the new construction is connected to the existing framing, the settlement of the new structure will cause the beams to tilt slightly. This may not have serious effects upon the floors, but it is likely to crack masonry walls and cause severe stresses in vertical bracing that joins the new and the old steelwork. If the two structures are isolated and if new columns are placed alongside the old ones, as shown in Fig. 10-17(*b*), this arrangement will have many obvious advantages. However, some means should be provided for preventing small offsets in the floors over which persons may stumble. Some type of adjustable or "floating" panel should be used at these junctions to attain a ramp effect.

It is obvious that driving piles alongside an existing structure like that of Fig. 10-17(*a*) is likely to disturb the piles and the soil under the building. When an extension is anticipated some time in the future, one might consider the following alternative procedures:

1. Drive the piles under the exterior wall of the original building as in Fig. 10-17(a), using enough of them for all probable future loads and pouring the cap or footing to deliver the present and the future loads to them. In the future, connect the framing to the existing structure, using corrugated or other relatively flexible siding.

2. Use a mat foundation or continuous narrow footings under the outer bay (at least), and cantilever the end wall, as in Fig. 10-17(c). In the future, drive the new piles and construct the addition as an isolated structure The advantage of this scheme is the fact that the future construction may be what and where it needs to be, and it is not affected by present wrong guesses.

3. Use the scheme in item 2 but drive at least the adjoining three or four rows of piles to serve as a buffer when the additional ones are driven. Preferably, these piles should not be capped until the future work proceeds, thus permitting additional piles to be driven and the footings to be designed when the complete requirements are known.

Of course, the trouble with all these suggestions is the fact that they require extra cost at the time of the initial construction. Many owners object to this.

10-11. Sequence of driving piles. As stated previously, it often happens that a large number of piles are to be driven for a new structure alongside an existing one, or that the new work is to be an extension of an existing building. Of course, disturbance of the old structure is to be minimized. How should the piles be driven and in what order? The answer to these questions depends upon the conditions at the site, possibly upon the type of pile to be used, and how they are to be driven.

Assume, first, that the project is the extension of a pier, that the plans of the ends of the new and the old structure are as shown in Fig. 10-18(a), that the soil profile is as pictured in Sketch (b), and that creosoted wooden piles are to be driven. In this case, it would seem that, if the pile driver were mounted upon a barge, the sequence of driving would not be very important as long as the work proceeds outward from the end of the existing structure, the order of driving being areas 1, 2, 3, 4, 5, 6, and 7, or 1, 2, 4, 6, 7, 5, and 3. If the pile driver is supported upon completed piling, the latter order may be preferable in order to minimize movement of the equipment. The movement of the mud and clay of the bottom will not be likely to disturb the existing or completed work seriously.

Next, assume the soil conditions pictured in Fig. 10-18(c), the existing structure being heavy and on end-bearing steel H piles or concrete-filled pipe piles. The pile driver must be supported upon piling or upon a heavy timber grillage or mat that is borne by the fill; for economy, it should utilize the new H piles as far as possible. The order of driving them depends very much upon what arrangement will best provide for a minimum of movement and shifting of the equipment. The general order might well be areas 1, 2, 4, 6, 7, 5, and 3 or 6, 4, 2, 1, 3, 5, and 7 of Fig. 10-18(*a*). The material displaced by the new H piles will not be great, and all piles

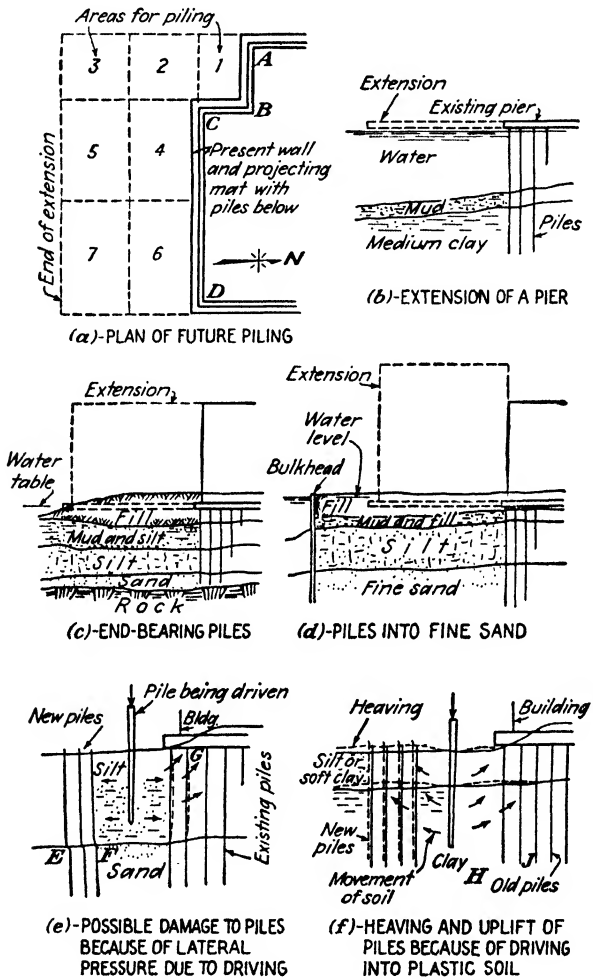


FIG. 10-18. Some problems encountered when driving piles alongside existing structures.

will reach a firm stratum of rock without excessively hard driving. Serious disturbance of the existing structure is improbable.

Assume a third case like that shown in Fig. 10-18(*d*). The original plant is on wooden friction piles; the extension, being much heavier, is to be supported upon thin shell cast-in-place concrete piles that are to support twice as much load per pile as do the wooden ones. The first plan considered is the driving of piles in the order of areas 3, 5, 7, 6, 4, 2, and 1 in order to relieve pressure upon the bulkhead due to displaced silt, and to compact the silt next to the old structure so that the piles in areas 6, 4, 2, and 1 might have an increased frictional resistance. If this is done, the piles driven in these last areas will displace considerable silt, and pressures may be exerted northward against the old structure and southward against the newly driven piles, as illustrated in Sketch (*e*). The former may bend the wooden piles severely and cause some upward pressure under the existing mat; it is unlikely to cause movement of the entire structure. On the south, the side of least resistance, the movement of the silt is very likely to displace the new piles sideways or to tilt them as shown in Sketch (*e*), breaking them at *EF*, the top of the sand. Because of this danger, the piles were driven in the order 1, 2, 4, 6, 7, 5, and 3.

Hard driving alongside an existing piled building is likely to disturb the present structure because of vibrations; it may also cause subsidence of the adjoining sandy or silty soil, and of the structure. Jetting of the piles may or may not be advisable. In the case of plastic soils, vibrations and movements of the supporting material may cause the adjoining piles to settle because of the impairment of their frictional resistance and of the shearing strength, cohesion, and compressive resistance of the soil upon which the piles react. Later on, the application of more loads upon the underlying strata below the piles, Fig. 10-18(*f*), may cause further subsidence because of compaction of this material.

When planning a pile foundation and when specifying the sequence of pile driving, a designer should bear in mind the nature and magnitude of pile-driving equipment. On a drawing it is easy to make circles to represent the piles without bothering to think how they will get there. Hence 3 piles in one group, 10 in another 20 ft. away, 6 in a third 25 ft. off in another direction, a row of 10 at 3-ft. spacing somewhere else, etc.—these may seem simple enough. With a floating driver, such scattered work may not be difficult; neither is it difficult on reasonably good soil. But in muck, soft clay, or loose and saturated silt, it is another matter. A pile driver weighs many tons; dragging it out of the mire is both difficult and costly. It may have to be supported on many temporary or permanent intermediate piles. No wonder then that the contractor's bid price for such work is high.

When piles are to be driven in entirely new work, the principal features to consider in planning the operations are the following:

1. Point of delivery of piles
2. Transportation of piles from stock to point of installation
3. Minimum temporary construction to hold equipment
4. Minimum movement of equipment
5. Parts needed first for construction of portions of superstructure
6. Ability to use driven piles as temporary supports for equipment

7. Portions needed as temporary supports for other purposes than holding pile-driving equipment
8. Adjustment of driving to supply of materials
9. Allowance of time for setting of concrete in piles
10. Use of available equipment if a special driver must be obtained for part of work but is not at hand, as for handling extra-long piles

If subterranean obstructions are known to exist in an area where piles are to be driven in large numbers, the planning as well as the sequence of driving may be affected. When piles are to be driven along old water fronts, thorough investigation first is good economy. Here is one example:

Several large steel oil tanks were to be built on hydraulic fill placed over a former shallow area at the margin of a harbor. The contents and the steel of each tank were to be supported by a concrete mat built on poured-in-place friction piles under the tank. A boring was made at the location of each tank, but no future trouble was disclosed. About half of the piles for the first tank had been driven when a serious obstruction was encountered about 12 to 15 ft. below the surface of the fill. After unsuccess-

FIG. 10-19. Raymond step-taper piles for the new boiler house at English Station of the United Illuminating Company, New Haven, Conn. Notice the filled piles, the splice in the shell in the foreground, the steam hammer driving the one at the right, and the timber trestle on wooden piles for the support of the pile driver. The wooden piles will be cut off and left in place. The soil is approximately 30 ft. of silt over fine and medium sand. *(Designed by Westcott & Mapes, Inc., Consulting Engineers, New Haven, Conn.)*

ful attempts to break through the obstruction by driving down the mandrel, the engineer ordered the rest of the piles be driven around the edge of the tank and thence the driving was to be conducted so as to work in toward the obstruction. The mat was then to be bridged over the obstruction. As it turned out, the area of the latter was too large to make this remedy practicable, but this was not realized until the surrounding piles had been driven. It was finally necessary to dig out the obstructing materials. This proved to be a slow and costly job because of the restricted space. The obstruction proved to be a lot of pieces of slabs of reinforced concrete that had been dumped there from a barge when someone demolished an old structure, and then they had been forgotten. After this, the engineer made careful explorations under the other tank sites. More obstructions were encountered under one of them, but these were dug out immediately and easily while they were accessible.

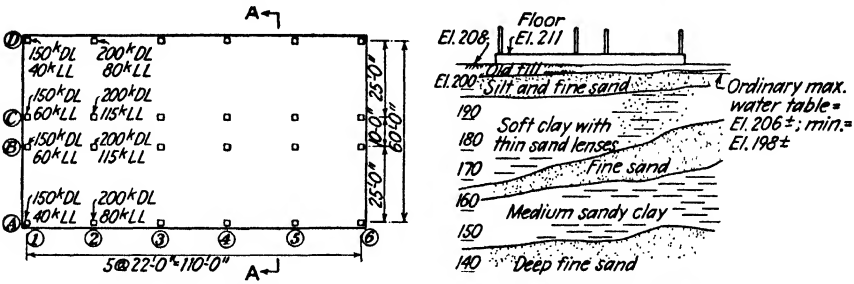
10-12. Piles vs. mats and spread footings. In engineering practice there are often situations in which there is a question as to whether piles are needed or whether the use of a mat or spread footings will be sufficient. Much may depend upon the correct answer. Each problem should be studied by itself. What is to be built? On what is it to be supported? How is it to be constructed?

One previously mentioned but useful concept for studying such problems is that of the equivalent uniform loading and its probable effects. For example, assume that nine wooden piles holding 20 tons each are to be used in rows 3 ft. 6 in. c.c. both ways under a footing. This is equivalent to a load of $40{,}000/3.5^2 = 3{,}300$ p.s.f. If the soil can hold this locally without squeezing out, the strata below may be able to do likewise because of the rapid distribution of the load by spreading from the footing. If the top stratum is not safe, then piles may be needed to transfer the forces down to a plane of suitable bearing value.

In order to illustrate what unit loads mean, assume that the footing referred to above is 10 ft. square with a load of $9 \times 40 = 360$ kips. Just below the footing this is equivalent to placing an imaginary column of earth 10 ft. square and about 36 ft. high on the soil. At 2:1 distribution, the assumed loaded area 10 ft. below the footing is 20 ft. square, and the imaginary superimposed load is a column of earth 20 ft. square and 9 ft. high. Similarly, at a depth of 20 ft., the fictitious applied load is a mass of earth 30 ft. square and 4 ft. high. This analogy may help to visualize what may happen to a structure.

How is one to start the solution of these problems? For example, assume that the owners of an industrial plant alongside tidewater wish to construct the small fireproof office building, shown in Fig. 10-20(*a*). It is to be founded upon the soils shown in Sketch (*b*). The average estimated dead weight of the structure, including the first floor, is 670 p.s.f. on the area of the building; the maximum probable live load, 300 p.s.f. There is no basement because of the high watertable, and steam will be piped in from the power plant. The building is to have a reinforced-concrete frame and brick or masonry-block walls. The typical loads for column footings are shown in Sketch (*a*).

Is a mat foundation advisable? In the first place, this seems to be a light building. Nevertheless, its weight is equivalent to a mass of earth approximately 10 ft. deep piled on the site. The top layer of silt and fine sand might not be harmfully compressible, but the thick tapered stratum of soft clay will compact considerably and unequally—tilting the building



(*a*)-PLAN OF COLUMN LOCATIONS (*b*)-SECTION A-A AND SOIL PROFILE

FIG. 10-20. Soil and superstructure data for small office building.

toward the left in Fig. 10-20(*b*) and perhaps cracking it. This is serious. Furthermore, other matters make a mat undesirable:

1. The floor should be an integral part of the mat for economy. This would require deep walls at the edges to get below the frost line, thus making doubtful the bearing of the mat on the fill adjacent to these walls.

2. The bays are rather large for effective spreading of the column loads over the intervening areas unless the mat is very thick or heavily ribbed.

3. There is so much load on the central columns that local sagging here may crack the structure.

4. This cannot be a floating foundation without a basement, and there is no necessity of building a basement for that purpose.

5. The structure is too valuable and too inflexible to put it on a mat of doubtful stiffness.

Is it practicable to use spread footings under the interior columns with footings and grade beams or heavy foundation walls under the exterior columns? No, for the following reasons:

1. The localization of pressure on the clay is likely to be greater, with larger differences in settlement.

2. The floor should be a self-supporting one connected to the column footings rather than independently resting upon fill, if uneven settling and cracking are to be avoided.

3. With a bearing value of 2 or 3 k.s.f. on the soil, the footings must be large anyway.

4. Again, the building should not be risked upon questionable supports.

If piles are then to be used, how should the construction be made? Here are some suggestions:

1. The variable watertable makes wooden piles undesirable unless the footings are very deep.

2. Poured-in-place concrete piles will be durable. Their lengths can be varied so that they penetrate a few feet into the deep layer of sand, but no deeper than necessary.

3. The soils are not really fluid, but the use of permanent thin shells on the piles seems to be desirable to make sure that the fine sand and silt will not run in and weaken uncased concrete.

4. The floor should be reinforced beam-and-slab construction that delivers its loads to the piles under the columns.

5. Place the bottoms of the footings at approximately El. 204 for exterior columns; 207 for interior columns.

6. Support outer walls on a deep grade beam between footings.

7. Use a combined piled footing for the pairs of interior columns.

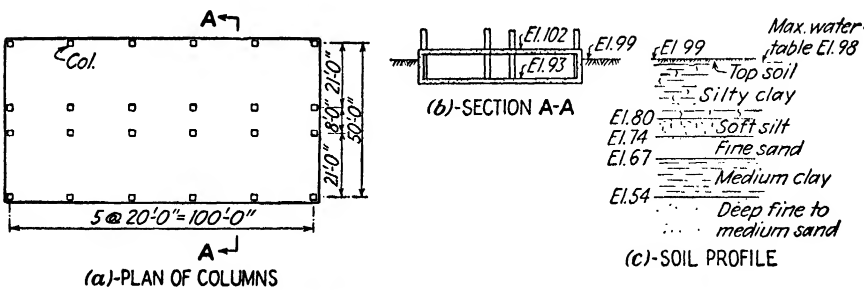


FIG. 10-21. Data pertaining to planning of foundations of a small office building.

Now assume that the small office building shown in Fig. 10-21 is under consideration. This is somewhat smaller than the preceding one in plan, but it has a basement. The soils are also different. Assume that, including the basement, the average dead load is now 750 p.s.f. and the live load is 400 p.s.f. What should be done in this case?

Piles might be used here—wooden ones if desired. They should be driven into the firm sand. A safe structure can be made in this manner with piles under the outer walls and under the pairs of interior columns, and with a good basement floor.

Could exterior foundation walls and interior spread footings be used safely? Perhaps so. At an allowable bearing value of 3 k.s.f., there would have to be a footing about 12 by 18 ft. under each pair of central columns. They are big. As for the pressure on the soft silt stratum under one of the central footings, its intensity will be approximately 1.2 k.s.f. This ought not to be harmful to the silt because it is trapped under a rather impervious layer, it is already compacted under a pressure of some 1.9 k.s.f., and it is thin so that the magnitude of the future compaction should not be large. Nevertheless, there will also be a tendency for the clay to compact near the center of the building. The basement walls and their footings can easily distribute the loads from the exterior columns.

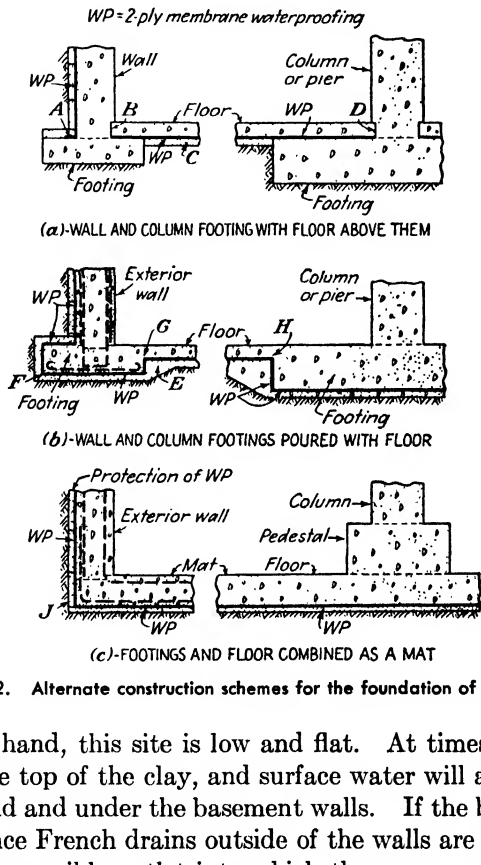


FIG. 10-22. Alternate construction schemes for the foundation of a building.

On the other hand, this site is low and flat. At times, the watertable will be above the top of the clay, and surface water will almost inevitably seep down around and under the basement walls. If the basement is to be kept dry and since French drains outside of the walls are assumed to have no reliable and accessible outlet into which they can empty, in this case, the construction should be made watertight. If so, the water pressure under the basement floor may be 300 p.s.f. or more. The floor should therefore be a structural waterproofed slab capable of resisting this pressure. This applies equally to the use of pile foundations.

To illustrate, assume that ordinary walls and footings are to be used and that they are to be constructed first. The waterproofing and floor slab are then to be built as shown in Fig. 10-22(*a*). It is obvious that water under pressure may leak through the construction joint from *A*, or under

B and *D*. The protection layer *C* will not stop the water. The pressure will tend to lift the slab; that is why the keys are used at *B* and *D*. These, however, are not very reliable as tight joints, especially around the columns or piers, and the slab will be weak in shear. Dowels to tie the slab to the footings are undesirable because they will interfere with the membrane. This construction is not desirable to resist real uplift.

Next, assume that the base course *E* in Fig. 10-22(*b*) is placed first and that recesses are made for the footings. The waterproofing is laid over the bottom to the edges *F*. Then the footings and floor slab are poured monolithically. The shear is transmitted from the slab to the footings at the sections *G* and *H*. The trouble with this is that the entire floor must be poured before any loads are applied. It will therefore try to act as a continuous mat, an action for which it is not designed.

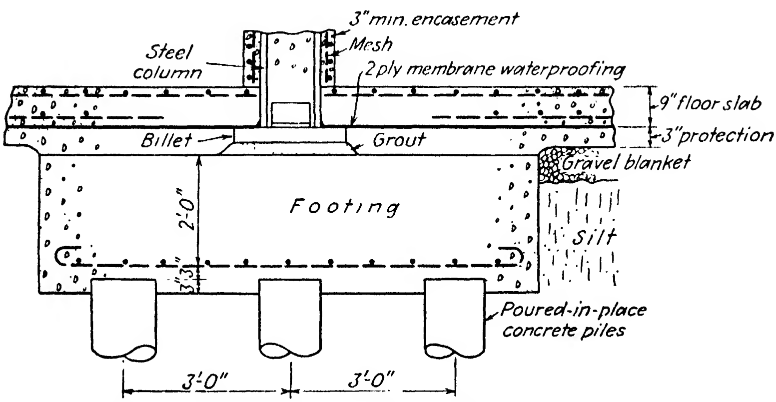


FIG. 10-23. Details of a 9-pile footing and waterproofed floor used at a multistory industrial plant.

Of course, this arrangement also brings heavy pressure on the waterproofing under the footings. One scheme for avoiding this is shown in Fig. 10-23. Here the loads are large, piles are necessary, and the hydrostatic uplift is small but not negligible. Notice how the steel base plate is set so that the waterproofing can be bonded to it. The upward reaction from the slab is resisted by the column encasement; the downward floor loads, by the top of the footing.

A glance at Fig. 10-21(*c*) shows that the excavation for the basement will be 5 to 6 ft. deep. The live load in the building may be looked upon as a temporary load that will not cause serious compaction of the silt and clay if the latter does not flow locally. Since the dead load of the structure practically equals the weight of the excavated ground, the structure will nearly be floating in the clay if the basement floor can make it act like a boat. It makes no difference to this floor whether the upward pressures are hydrostatic or are upward reactions from the earth. At least, the floor must resist the former. If it can do this, it will, as stated previously, try to act as a continuous mat in spite of the designer's wishes. Therefore, it seems desirable to design the floor to resist these forces, and to use construction like that of Fig. 10-22(*c*). In this scheme, the projecting footing beyond *J* is eliminated, thus simplifying the waterproofing problem. With this arrangement and the utilization of the "floating" foundation, it seems that piles are unnecessary. Apparently, this principle of a floating foundation was used successfully for the Loteria Nacional Building in Mexico City.[1]

Of course, if the water could be removed by trustworthy drainage outside the walls, the ordinary walls and footings might be satisfactory for this building. However, if the loads were considerably heavier, the footings would become so large that piles would probably be advisable.

Now suppose that the top stratum was weak silt and the next one clay, in Fig. 10-21, and that the watertable was at El. 88. Then the problem of water pressure would not exist, the soft silt might flow or compress under the footings, and the use of piles would be advisable, although the floating foundation would still be worthy of consideration. If the loads were considerably heavier, probably the piles should be used without question.

Consider an entirely different situation. An industrial company had built some large oil tanks along the Northeastern seaboard. A large new tank was later added to the tank farm. It was near the shore so that part of the concrete floor or mat rested upon excavated ground whereas the outer portion was upon a few feet of fill that seemingly covered softer material formerly the edge of a muddy flat. The steel walls were completed, and the tank was filled with water to test for leakage. Suddenly one side of the tank failed. The water gushed down and over the surrounding dike and flooded an adjoining pump house. Apparently, this short-circuited some of the motors and caused explosions of gasoline that resulted in a very disastrous fire, with loss of life as well as of property. There is some disagreement regarding the cause of the failure, but it seems probable that it was due to the distortion of the tall stiff wall produced by unequal settlement of the portion of the filled area. This case is one in which the mat cannot do any real spreading of loads. It is merely a sort of pavement that necessarily accompanies any settlement of the ground under it.

Perhaps there are two lessons to be learned from the preceding illustration:

1. One should obtain a scale on the magnitudes of the total loads applied to the soil. An oil tank 150 ft. in diameter and about 30 ft. high may cause an average unit pressure of about 1,900 p.s.f. This seems small. However, the total load is something like 33,500,000 lb., or 17,000 tons. What could this do to cause movement of weak cohesive soil?

[1] *Engineering News-Record,* Dec. 12, 1946, p. 95.

2. Small unequal settlements, or a small change in the angle of settlement, may cause very severe strains and accompanying stresses in stiff structures. The wall of the tank referred to is a sort of stiff vertical plate girder in a curved position. Ordinarily it has to resist merely the ring tension produced by the fluid pressure. However, an unequal settlement, as shown in Fig. 10-24(*a*), may rupture the junction of the bottom and the wall, or it may deform the wall excessively. A change in angle like that in Fig. 10-24(*b*) is likely to tear the top apart. Supposedly, this is what caused failure in this instance. Therefore, when settlements of the various parts of a structure are likely to be unequal and when they may cause serious

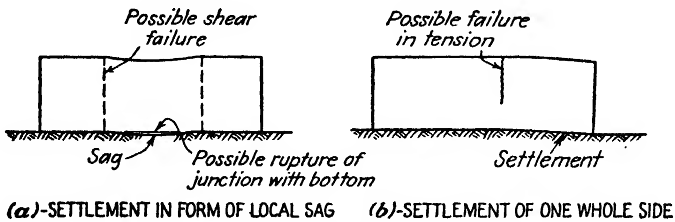


FIG. 10-24. Character of possible damage to an oil tank caused by uneven settlement.

trouble, it is best to use piles or some other type of substructure known to be trustworthy, but the structure should be considered as a unit. Local hard spots may hurt a structure somewhat as a marble hurts one's bare foot when he steps on it in the dark.

Cost estimates of alternate plans should be made before one decides upon the solution for a specific problem. One should not jump to conclusions too quickly.

10-13. Vertical piles subjected to horizontal forces. There are many special features that should be borne in mind when one plans pile foundations. By necessity, the information herein is largely qualitative because quantitative data depend upon the soils, piles, forces, and dimensions involved.

If the pile shown in Fig. 10-25(*a*) has a continuously applied horizontal force H acting as in (*b*), and if the pile is surrounded by weak cohesive soils or muck, it will tend to bend as shown to exaggerated scale in the latter sketch. The pressure diagram may be of the character shown in (*c*). The pile will bear against the soil on the left side from A to some point D. Below D a small pressure on the right will resist the tendency of the pile to rotate counterclockwise and to curve. The soil may yield gradually under the high-pressure AC so that the pressure diagram may more nearly approach $C'D'B$. If the soil is surrounding the full length of the pile, it can resist small lateral loads unless the ground is practically fluid. But when the pile projects far above the soil, as in Fig. 10-25(*d*), the resistance of the soil and the bending strength of the pile may become important.

In actual practice, the lateral force will be resisted as efficiently as possible, *i.e.*, as near the top as the strength and stiffness of the soil and pile permit. Quantitative values of pressures and bending moments are difficult to ascertain. Cylindrical reinforced concrete piles and steel H piles have uniform stiffness along their lengths whereas this property varies in tapered piles. If the pile is embedded in fine sand or firmer soils, the point of maximum bending may be 5 ft. below the top of the soil; if in soft clay, 10 ft.

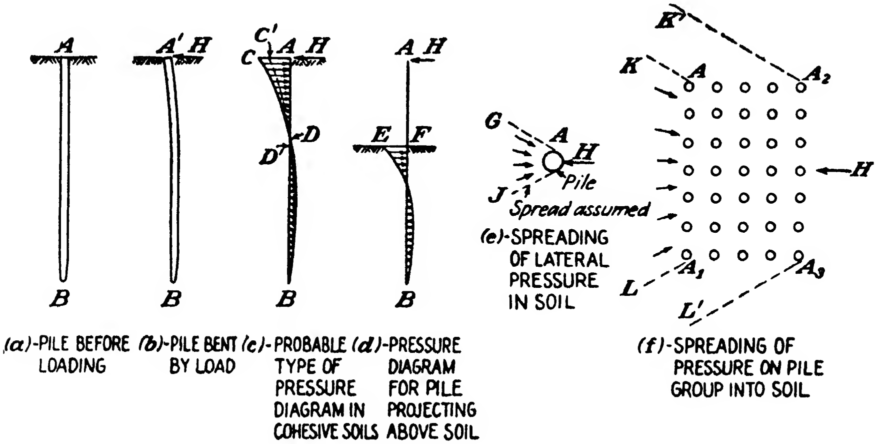


FIG. 10-25. Vertical piles subjected to horizontal loads at their tops.

below; if in mud or muck, a few feet below the top of the highest stratum that is soft clay or better.

For a single pile, the abutting resistance of the soil may be much greater than that caused by a strip of earth equal to the width of the pile. The pressure may spread through the ground as indicated by GAJ of Fig. 10-25(e) if the soil is not too weak, causing a very considerable resistance to finite displacement.

On the other hand, the pressure from a row of piles AA_1, Sketch (f), will spread sidewise through no more than some volume represented by KAA_1L, whereas a large group will not affect much more soil than does the single row, as pictured by $K'A_2A_3L'$. This shows that the results of a lateral pulling test on one pile may not yield reliable data for the resistance of a group of piles unless the spacing is large or the soil is fairly firm or almost fluid near the top. It is advisable to assume that the resisting mass of earth for a closely spaced group of piles in reasonably resistant soil is a vertical portion not more than 6 to 10 ft. wider than the distance center to center of piles across the group perpendicular to the active horizontal force—AA_1 of Fig. 10-25(f). The abutting power of this earth may then be estimated. The upper 10 to 15 ft. should be able to resist the applied force H.

No great accuracy should be expected from such computations of lateral resistance. One should be very conservative in design. Seldom will the situation be dangerous when the pile is fully embedded in soil of moderate stiffness. On the other hand, vertical piles should not have permanent horizontal forces applied to them when there is any doubt of the rigidity of the surrounding soil. Temporary wind and live-load shears of small magnitudes are usually harmless if the soil is at least as good as moderately compacted silt or rather soft clay.

Groups of piles may be lashed together, as in the case of dolphins, to resist large horizontal forces. If the piles can slip past each other, the longitudinal shearing resistances will be inadequate to make the group act as a large single member; it will be only as good in bending as the sum of the strength of the individual piles. Furthermore, the whole group will be no better than the lateral resistance of the soil in which the piles are driven.

Vertical piles that are braced together to form towers or bents may have considerable lateral strength initially. However, such bracing members and their connections are often difficult to install and are not likely to be permanent unless they are well protected and maintained.

If the horizontal forces acting on a group of piles are caused by the tendency of the surrounding soil itself to move laterally, the piles can offer little or no resistance since they depend upon the supporting soil for their own strength. If the piles go through a deep layer of unstable soil into a stable sand stratum, they will usually tilt or be broken by the deformation caused by the movement of the top layer.

A good example of the fact that piles embedded in a deep plastic soil will merely move with that soil is the case illustrated in Fig. 10-26. When this viaduct was planned, it was considered desirable to terminate the superstructure near the flare in the roadways shown. This required an extensive fill a little over 30 ft. deep above the original ground, but it would avoid the use of complicated and costly structures. The soil was a layer of peat 8 to 10 ft. thick over a very deep stratum of fairly soft clay.

The main portion of the viaduct consisted of a series of rigid-frame units like *EH* in Fig. 10-26, with hinged suspended spans between them. Toward the abutment, however, the structural character but not the appearance of the viaduct was changed, as shown in (*b*). This permitted movement of the abutment *A* and footing *D* to occur without injury to the girders because of the roller at *A* and the hinges at *B*, *C*, *D*, and *E*.

The peat was first removed from the area to be covered with the fill. The latter was then placed and compacted. Steel pipe piles filled with concrete were installed and capped by the concrete footing and bridge seat of the abutment. These piles were used merely to ensure that future compaction of the fill itself would not cause settlement of the abutment. However, the clay was expected to settle and possibly to flow laterally.

The backwall of the abutment was made of bolted timbers so that it could be rebuilt later if necessary. The bearing at A was a geared roller on an excessively wide seat with provisions for jacking up the girders and raising the bearing surface. Jacking and adjustment were also provided for at D.

After about three years, the abutment at A had settled approximately 16 in. downward and a few inches backward. Of course, the piles were carried with the clay in which they were embedded. As expected, the greatest settlement occurred under the greatest mass of the fill, hence the

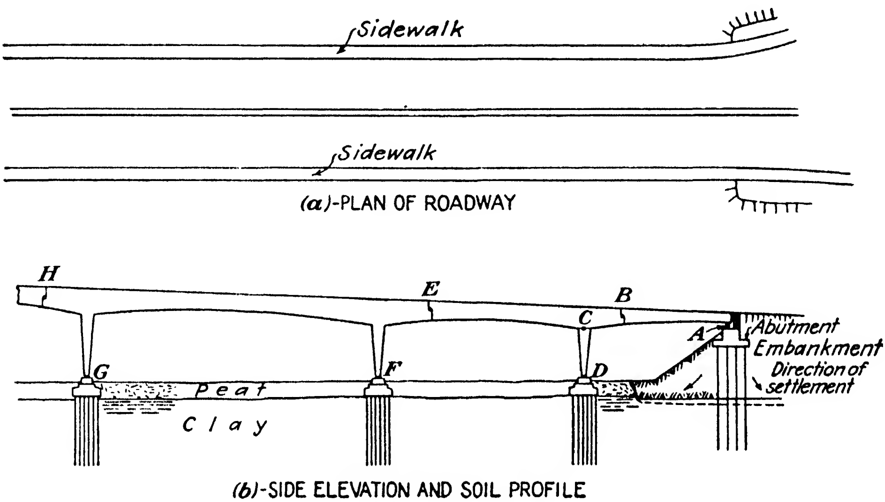


FIG 10-26. A viaduct planned for anticipated settlement of abutment.

backward rotation of the top. Some movement of the clay toward D also occurred. The first revision of the abutment was made as intended, and the roadway on the fill was raised. Undoubtedly, other alterations will be required in the future, but the magnitude of the settlement per year should decrease progressively.

10-14. Batter piles. When large horizontal forces are or may be applied to the tops of piles, the use of batter piles is generally desirable. If driven to the same horizontal level as the tips of vertical piles, batter piles may be relied upon to resist the same longitudinal load as the vertical ones, provided, of course, the soil conditions are the same.

Assume that Fig. 10-27(a) shows a cross section through a reinforced-concrete retaining wall or bulkhead supporting the edge of a yard area at an industrial plant near the water's edge. The horizontal component H of the earth pressure P tends to tip the wall about point A and cause it to slide toward the left. If piles are used as shown in Sketches (a) and (b), horizontal movement of the wall will cause the vertical piles to bend or tilt leftward whereas piles d, e, and f will tend to be forced farther into the sand or to tip about their ends. It is obvious that only the horizontal component of the resistance of the batter piles to penetration should be relied upon to counteract the thrust H.

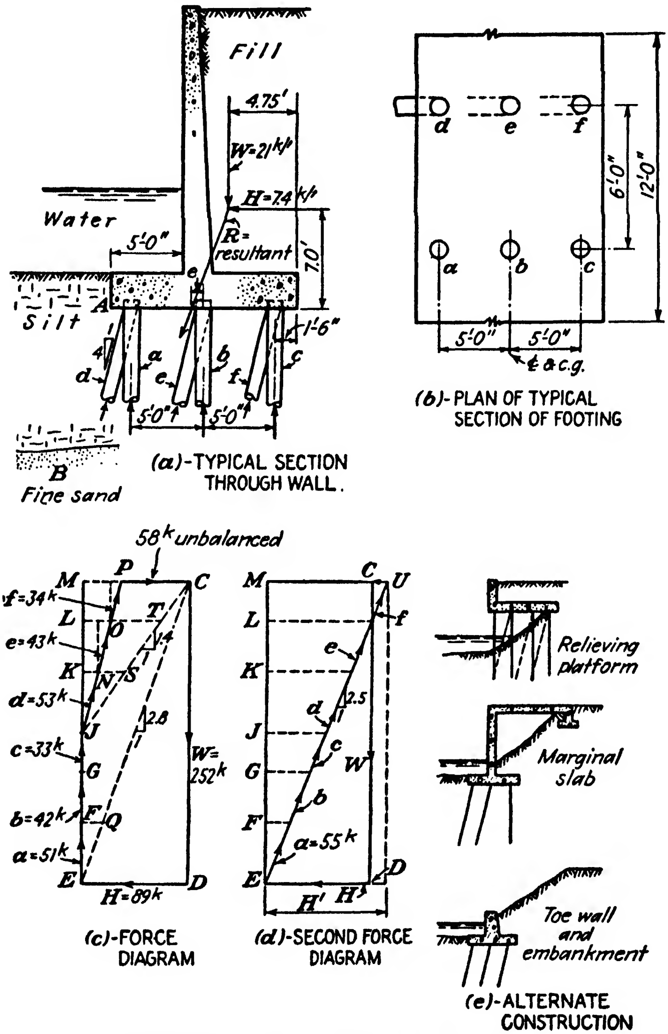


FIG. 10-27. Analysis of loads on batter piles.

When using a combination of vertical and batter piles, one should remember that the structure will try to act as a unit. One should not assume that the vertical piles resist all the vertical loads whereas the batter piles resist all the horizontal forces. Referring to Fig. 10-27(a) again, it may be that the applied loads cause piles c and f to produce a small upward vertical or even a downward reaction; piles b and e, an upward resistance; piles a

and d, a large upward reaction. In each case, the forces will be assumed to be parallel to the longitudinal axis of the pile. It is obvious that the batter piles are the effective ones in resisting H, but they are not unaffected by the vertical load W.

The method used to compute the forces upon batter piles should be simple, logical, and easy to apply. Its results should be reasonably near reality. Tedious computation amidst so many uncertainties is not necessary, nor are its results more accurate than the assumptions of loads, soil pressures, soil resistances, etc., upon which it is started. The following procedure is therefore suggested:

1. By approximations or guesses obtain a trial design for the structure. Give all dimensions and assumed loads.

2. Compute the center of gravity and moment of inertia of the pile group, or a typical portion of it, as though all piles were vertical.

3. Compute the overturning moment and eccentricity of load at the plane of the tops of the piles, or at the bottom of the footing.

4. Compute the vertical load on each pile.

5. Draw a force diagram showing the horizontal force and the computed vertical loads on the piles.

6. Draw the hypotenuse of each right triangle of which the computed vertical force on a batter pile is the vertical component, sloping it at the assumed batter of the piles. If these, when drawn in the force polygon, close the polygon, the computations may be assumed to check. If the batter of the piles is not assumed, the slope needed to close the force polygon may be assumed to be the desired batter.

Apply this to Fig. 10-27 as an illustration. Since the pile rows are assumed to be 6 ft. c.c., assume a slice of wall 12 ft. wide as a typical case. If the loads and their positions are as shown in (a), $H = 12 \times 7.4 = 89$ kips and $W = 12 \times 21 = 252$ kips. Neglecting the moments of inertia of the individual piles about their own centers,

$$I = 4 \times 5^2 = 100 \text{ pile-ft.}^2$$

The eccentricity of the load is

$$e = 3.25 + 7 \times \frac{89}{252} - 5 = 0.72 \text{ ft. left of pile } b$$

$$M = 252 \times 0.72 = 181 \text{ ft.-kips}$$

$$p_a = p_d = \frac{252}{6} + \frac{181 \times 5}{100} = 42 + 9 = 51 \text{ kips}$$

$$p_b = p_e = \frac{252}{6} + \frac{181 \times 0}{100} = 42 \text{ kips}$$

$$p_c = p_f = \frac{252}{6} - \frac{181 \times 5}{100} = 42 - 9 = 33 \text{ kips}$$

The force diagram is drawn in Fig. 10-27(*c*). Line *JP* represents the inclined reactions of the batter piles since it is drawn parallel to them. The magnitudes are scaled. Line *PC* represents the portion of *H* that is not resisted by the horizontal components of the batter piles. This might be resisted by bending in all six piles if it is withstood at all. Since the silt around the piles will tend to be forced out by the pressure of the fill behind the wall, it is unwise to place any dependence upon the silt to support the piles against lateral bending. Therefore, something else should be done.

Line *JC* of Fig. 10-27(*c*) shows the batter needed by the three piles if they are to resist the entire thrust *H*. On the other hand, *EC* shows the slope required if all six piles are battered, and *EQ* gives the resultant thrust in pile *a*. However, if all piles are battered, the weight of the wall alone before the fill is placed will cause the structure to tend to tip backward and bend the piles. This silt may be strong enough to resist this before it is loaded otherwise, but such a condition is not generally desirable when the soil is nearly fluid.

Now suppose that all six piles of Fig. 10-27(*b*) were driven at a batter of 2.5:1. Incidentally, a slope of 2:1 is a practicable limiting angle for convenient driving. Line *EU* of the force diagram in Sketch (*d*) represents the reactions of these piles. The distance *CU* is a theoretical excess of pile thrusts above the value of *H*. This, however, is not harmful. All it does is to tend to push the wall against the fill until enough passive pressure is developed to cause equilibrium. Therefore, if the piles can resist safely the maximum active pressures for any combinations of loading, a smaller active load will merely cause the development of enough of the tremendous potential passive resistance to produce equilibrium.

Increasing the weight of the wall will merely require more piles. Something has to resist *H*. It therefore seems best to modify the construction by using a relieving platform, a heavy concrete slab, or a smaller wall and sloping embankment, as indicated in Fig. 10-27(*e*).

As another illustration, assume the bridge pier shown partly in Fig. 10-28. Sketch (*a*) shows a typical row of piles; (*b*), a plan at one end. Assume that all outer side piles are battered as shown, that the four at each end are battered transversely, and that all others are vertical. For a typical row, $W = 290$ kips, $M = 230$ ft.-kips, and a shear S at the bottom of the footing = 5 kips. Then,

$$I = 2(1.5^2 + 4.5^2 + 7.5^2) = 158 \text{ pile-ft.}^2$$

$$p_a = \frac{290}{6} + \frac{230 \times 7.5}{158} = 48.3 + 10.9 = 59 \text{ kips}$$

$$p_b = \frac{290}{6} + \frac{230 \times 4.5}{158} = 48.3 + 6.5 = 55 \text{ kips}$$

$$p_c = \frac{290}{6} + \frac{230 \times 1.5}{158} = 48.3 + 2.2 = 51 \text{ kips}$$

$$p_d = 46 \text{ kips} \qquad p_e = 42 \text{ kips} \qquad p_f = 37 \text{ kips}$$

The force diagram is shown in Fig. 10-28(c), where CO is the sum of the computed vertical components. Lines CJ, JK, KL, LM, MN, and NP represent the reactions of the piles. Line PA is negligible. If it had appreciable magnitude, it would indicate a desired change in the batter. However, small amounts of unbalanced shear could be resisted by the piles in bending or by lateral bearing on the soil.

The end row of piles can be treated the same in the analysis as any interior row. The reactions on the four central battered piles will be slightly

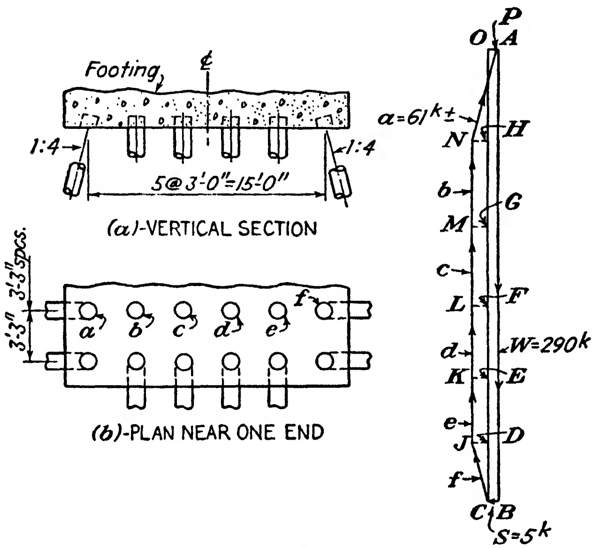


FIG. 10-28. Piles supporting a bridge pier. (c)-FORCE DIAGRAM

larger than the computed vertical components. The piles at the two ends counteract each other's horizontal components so that no harm results. In some cases, it may be advisable to use more batter piles sloping transversely with respect to the bridge.

As a third illustration, assume that Fig. 10-29(a) pictures a typical row of piles under the footing of a rigid-frame rib in a hangar. The concrete ends at ABC. Extend AB to D. The intersections of this plane with the centers of piles a and b are shown. Assume them to be in these positions. Then, about the center of the group,

$$M = 60 \times 6 - 200 \times 2.5 = -140 \text{ ft.-kips}$$

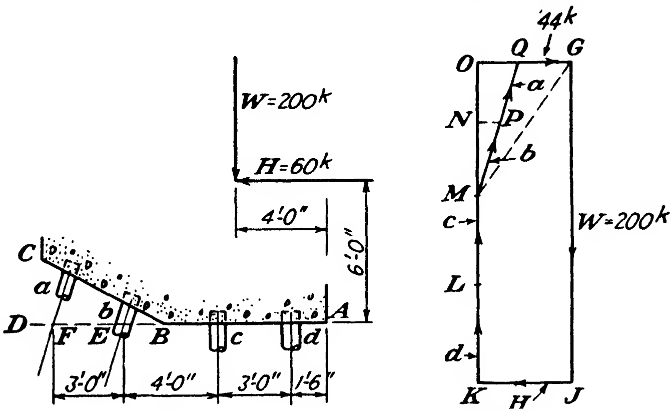
$$I = 2(2^2 + 5^2) = 58 \text{ pile-ft.}^2$$

$$p_a = \frac{200}{4} - \frac{140 \times 5}{58} = 50 - 12 = 38 \text{ kips}$$

$$p_b = 50 - \frac{140 \times 2}{58} = 50 - 5 = 45 \text{ kips}$$

$$p_c = 50 + 5 = 55 \text{ kips} \qquad \text{and} \qquad p_d = 50 + 12 = 62 \text{ kips}$$

The force diagram is then constructed in Sketch (*b*). Line *QG* shows that approximately 44 kips of the thrust *H* remain to be counteracted. Piles *a* and *b* might have their slope increased to match line *MG* if this is not too steep for practicable driving. The footing might be redesigned to set *W* back farther, thus increasing the forces and horizontal components in piles *a* and *b*. However, this would avail little unless pile *c* were battered also. Because of variations in live load and wind reactions, it seems desirable to keep pile *d* vertical. Of course, passive pressure behind the



(*a*)-FOOTING AND ACTIVE FORCES (*b*)-FORCE DIAGRAM

FIG. 10-29. Bearing of a rigid frame.

footing would help also, but a footing for a rigid frame should be designed so that it is entirely trustworthy.

10-15. Piers. Many piers are founded upon piles; in fact, most of them are. Piers (and many bulkheads) are not so much a problem of pile foundations as they are of pile structures. Here the piles are primarily stilts or columns to reach from the pier deck down through air and water to the underlying soils. That these soils are often silt, clay, and fine sand is to be expected because of the geology and topography of the sites. Where bare rock or a thin gravel stratum over rock exists at a proposed pier, some construction other than piles alone is likely to be more economical, and also desirable to attain lateral stiffness.

The live loads on pier decks are likely to be heavy in terms of pounds per square foot—500 to 1000 p.s.f.—but not in terms of the development of large loads per pile. However, columns of double-deck transit sheds, warehouses on the piers, truck passageways, and railroad tracks usually require strong pile foundations. The planning and design of such structural foundations are similar to others except for the fact that the piles are more like long columns, they may be subjected to considerable lateral bending, and they are more exposed to disintegration.

The problems of pier construction are largely those of practical structural framing rather than of the attainment of adequate support. Steel piles may be useful when the piles must be very long; creosoted wooden piles, when flexibility against lateral impacts from ships is necessary; precast concrete piles, when strong column and beam action are essential but the

FIG. 10-30. Driving steel H piles for pier 1S of the Chesapeake City Bridge, Chesapeake City, Md. Notice the spacers tack welded along the steel bracing to assist in spotting the piles. The steam hammer is in position for driving. Note that the pile has been moved the correct distance away from the spud (at the right) to align with the hammer anvil. This is accomplished when the pile engages the foot block or spacer at the tip of the spud. *(Designed by Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y. Built by Fehlhaber Pile Co.)*

lengths are not great. When rock is easily accessible and a heavy structure is a part of the pier, it may be advisable to use drilled-in caissons with steel H's, large pipe piles filled with concrete, or even a number of 4- or 5-ft. caissons placed under the columns and resting on the rock.

There are so many special details of construction used for piers that minute descriptions of them here seem unwarranted. Some are illustrated by the accompanying drawings. Local conditions, the materials available, the desired permanence of the structure, and the loads to be supported are likely to influence the choice of the general type of pier construction. Piers made of precast concrete piles and concrete decks are especially worthy of consideration because of durability, fire resistance, low maintenance, and long-term economy. Precast members in the decks may also save considerable in the cost of forms.

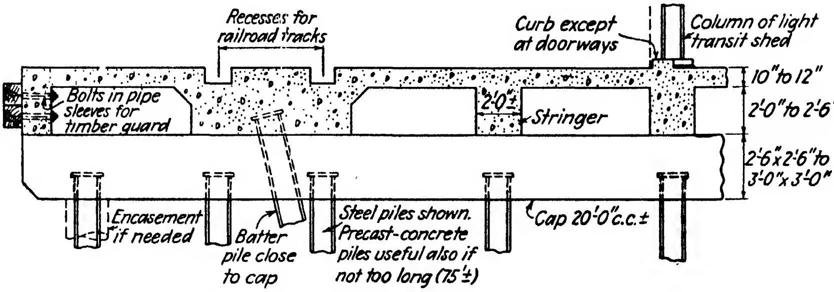
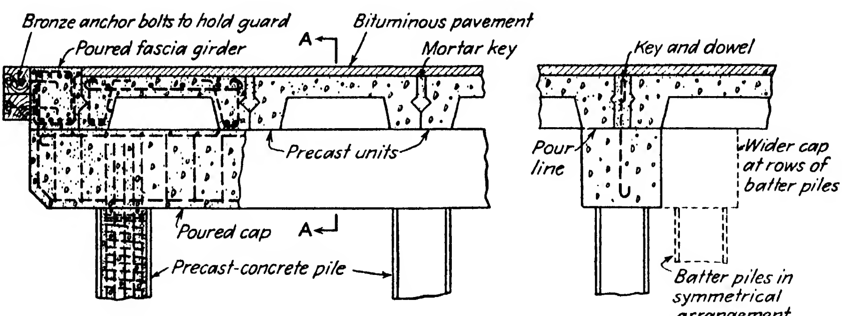


FIG. 10-31. A poured-concrete deck for pier on long steel piles.

A pier should be stable against horizontal forces. These forces may be caused by wind, waves, ships, ice, or floating debris. Bracing of the structure near the tops of the piles may be sufficient when the latter are firmly embedded in deep stiff soil. In other cases, batter piles transversely and longitudinally may accomplish this. They may be placed between or next to the rows of vertical piles. In any case, they should be attached firmly and arranged so that they do not cause unbalanced horizontal thrusts against the deck when vertical loads are applied.



(a)-PARTIAL TRANSVERSE SECTION (b)-SECTION A-A

FIG. 10-32. Suggestion for reinforced-concrete pier with precast deck units.

10-16. Bulkheads on piles. The problem of earth pressures against high bulkhead walls has already been illustrated somewhat in connection with Fig. 10-27. Heavy masonry may be suitable on rock, hardpan, gravel, or even on firm and coarse sand but not on weak cohesive soils where piles are needed. In these last cases, the relieving-platform type of construction is useful.

Several varieties of relieving platform are shown in the accompanying illustrations. These are intended to show many different details, and they

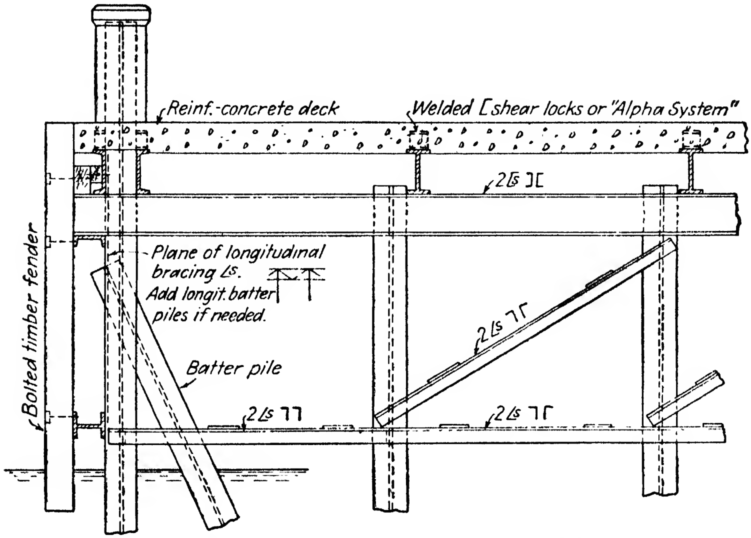


FIG. 10-33. Some construction for a structural-steel pier with concrete deck. (Double pile bents to be used at transverse expansion joints. Copper-bearing steel may resist corrosion.)

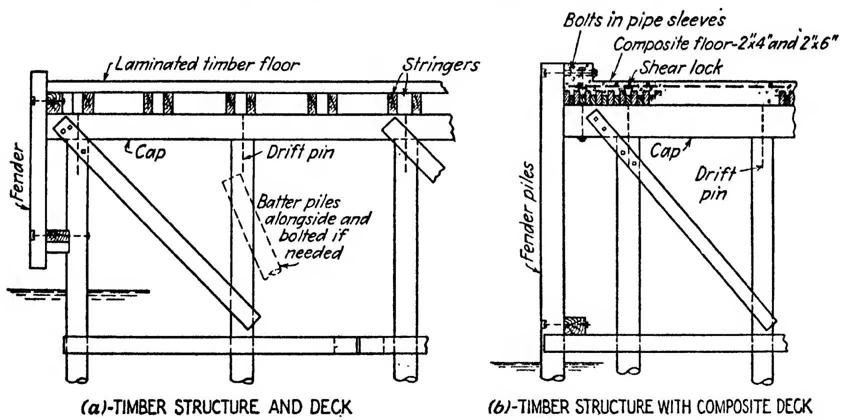


FIG. 10-34. Some construction for wooden piers.

may be combined in various ways. The use of the earth fill in Fig. 10-36 requires more piles per square foot to support the vertical load than do Figs. 10-35 and 10-37, but this type is convenient when utilities and railroad tracks are to be placed in or on the fill. On the other hand, the plat-

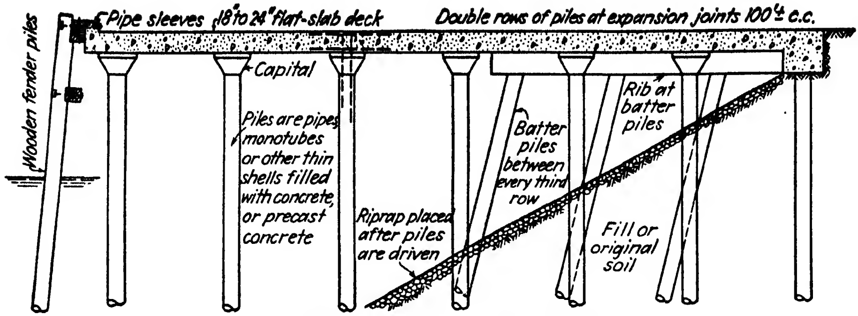
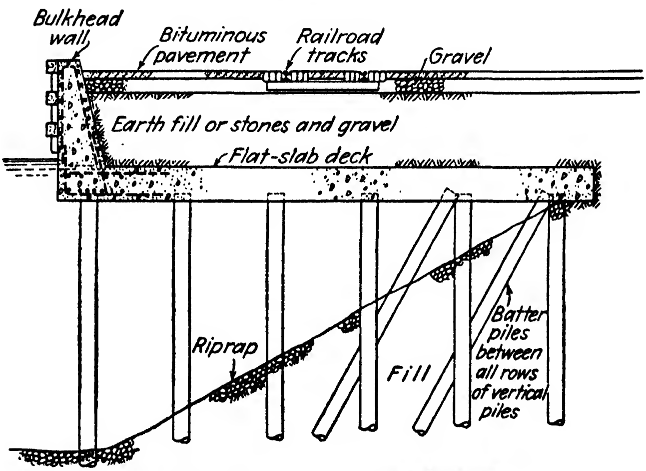
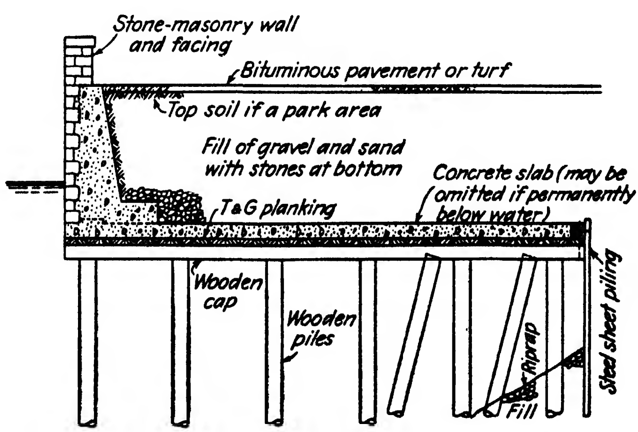


FIG. 10-35. A flat-slab relieving platform.



(a)-CONCRETE-AND-EARTH-FILLED CONSTRUCTION



(b)- CREOSOTED WOODEN PILES AND DECK WITH CONCRETE AND EARTH COVER

FIG. 10-36. Relieving platforms with earth fill.

form in Figs. 10-35 and 10-37 must be wider for a given freeboard and depth of water because the slope of the earth below starts almost at the surface of the ground whereas, in Fig. 10-36, it starts from 10 to 15 ft. below grade. In general, the use of the earth cover is the more economical arrangement.

The batter piles are usually in rows placed between the vertical ones. They should not project into the water in front of the bulkhead where they might interfere with future deepening of the waterway. Therefore, their tops are near the back edge of the deck slab. In this construction, the

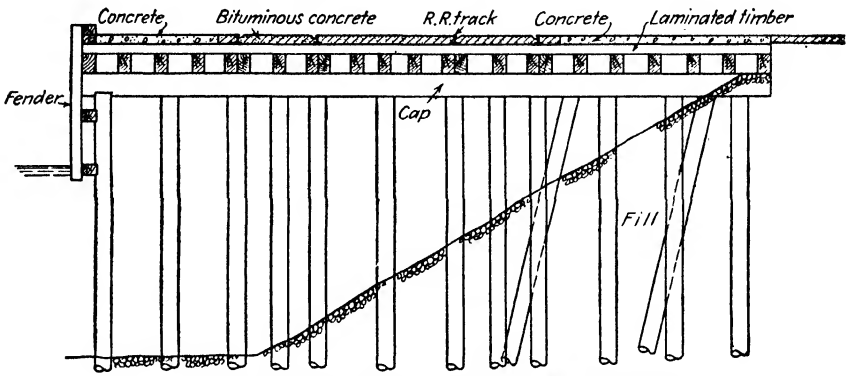


FIG. 10-37. A creosoted-timber bulkhead of relieving-platform type.

batter piles are to resist any forces tending to move the structure waterward, *e.g.*, wind blowing against a ship tied to the wharf, and active earth pressure. The abutting resistance of the soil will prevent landward movement. The vertical (plumb) piles, especially those embedded deeply in the sloping earth, generally have considerable lateral stiffness, especially when the slope is considerably flatter than the angle of repose, when the fill is composed of gravel, and when it is armored deeply with riprap.

Fenders are generally desirable to prevent damage to ships when they strike or scrape against the bulkhead. It may also be advisable to extend the fenders or a wall down below low-tide level in order to keep small boats and driftwood from catching under the structure.

One danger to be guarded against is the placing of fill on silt or clay that may move slowly waterward because of the weight and the small cohesion and low angle of repose of these soils when saturated. Tides and waves may also tend to move the soil away and destroy the resistance to lateral movement of the structure. Gravel and riprap coverings help to reduce these effects, but it may be advisable to excavate dangerous soils before the structure is built.

The sheet piling at the rear of the bulkhead in Fig. 10-36(*b*) is used to shorten the necessary width of the platform, especially when the waterway is deep. This adds to the outward thrust. However, if the piles penetrate well into sand and if the fill is made of good material well compacted, the batter and plumb piles generally permit the use of a moderate pressure against the sheet piling.

Figure 10-38 pictures a type of relieving platform with a sheet-pile wall at the front. Some advantages of this construction are the following:

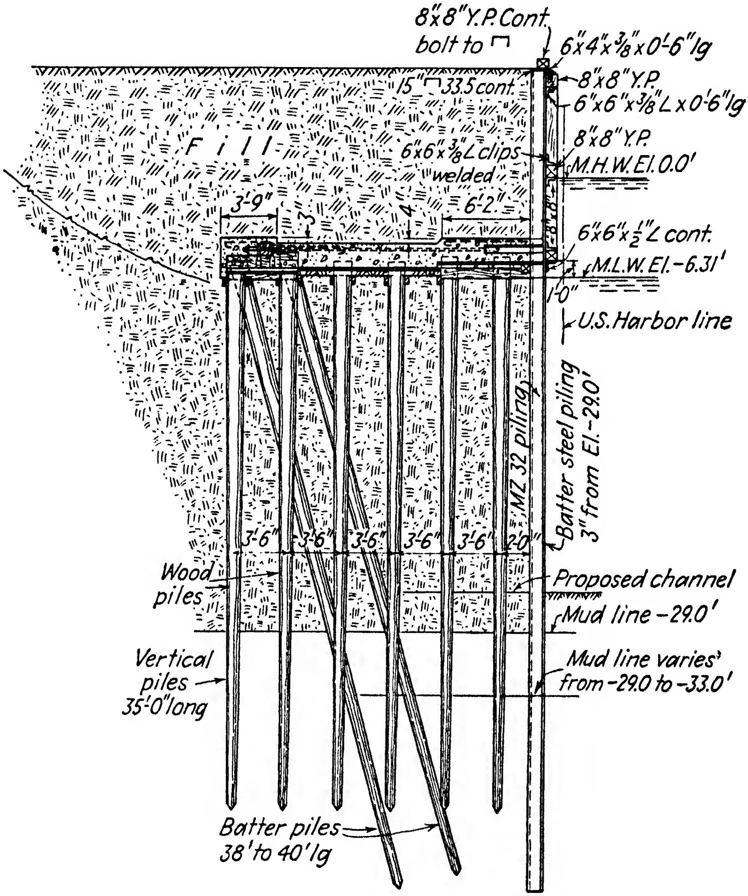


FIG. 10-38. A type of relieving platform with sheet-pile face and batter piles. (Courtesy of Arthur G. Beaulieu, Bulkheads: Their Purpose—Design and Construction, 56th Annual Report, The Connecticut Society of Civil Engineers, 1940.)

1. Wooden piles are embedded in fill or soil, kept continuously wet, and protected from marine borers.
2. Sheet piling forms the front wall at the top.
3. Sheet piling can be driven first, then used as a cofferdam for the remainder of the construction.

4. If the soil or fill is firm enough, the concrete deck can be poured directly upon the ground without expensive forms or a timber platform.

A sheet-pile and anchorage bulkhead is shown in Fig. 10-40. This is a practicable structure when the subsoils are firm enough and when the fill is not too deep. It is not generally suitable for large ships and deep waterways. It consists primarily of a continuous wall of deep-arch or Z-shaped steel sheet piling attached to steel tie rods uniformly spaced. These rods are connected to a bolted or welded wale placed so that the cantilever

FIG. 10-39. Tied sheet-pile bulkhead construction at English Station of the United Illuminating Company, New Haven, Conn. The old bulkhead is shown at the right with the tie rods anchored to it. The new sheet piling is at the left with timber wales and pile bents to line it up. The tie rods are adjusted by turnbuckles and are permanently supported upon timbers and piles as shown. Before placing the fill the tie rods will be boxed in concrete for protection. ***(Courtesy of Westcott & Mapes, Inc., Consulting Engineers, New Haven, Conn.)***

effect of the top AB reduces the maximum bending moment in the portion BD. It is very important that the embedded toe CD be sufficient for the soil to develop the necessary passive resistance to the outward thrust of the sheet piling.

The anchorage of the tie rods is very important. Sketch (a) shows the use of a continuous wall on vertical and batter piles. If the line CE is at the angle of repose ϕ of the saturated fill and soil, the wall itself cannot do much to anchor the bulkhead because it will push against the soil which is, in general, pushing against the sheet piling. The plumb piles will tend to resist being pulled out and tilted, and the batter piles will be very effective. If a wall is used alone, as in (c), it should be placed considerably back of line CE. Other details of anchorages are shown in (b) and (d). The latter, too, should be placed 10 to 15 ft. beyond the plane of the angle of repose if this sheet-pile wall is to develop the necessary passive resistance.

The tie rods should be adjustable by means of turnbuckles in order to line up the bulkhead properly. These rods generally need intermediate piles for support against the weight of the soil above them. In one case this was not done, in spite of the engineer's advice. Furthermore, the fill

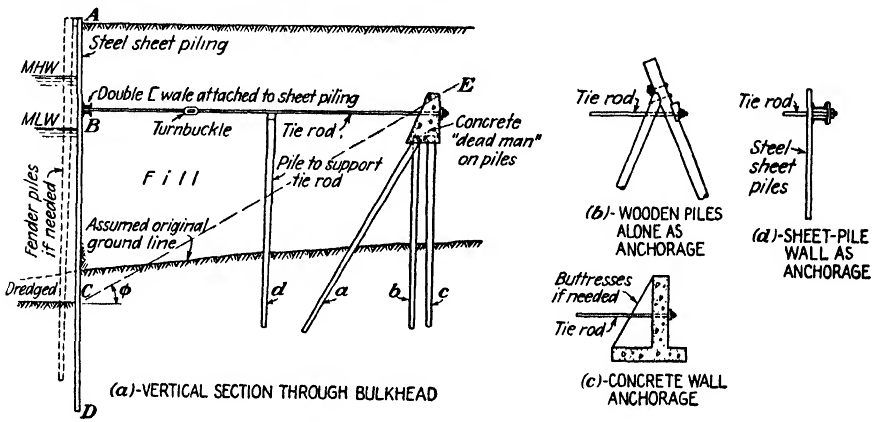


FIG. 10-40. Sheet-pile bulkhead construction.

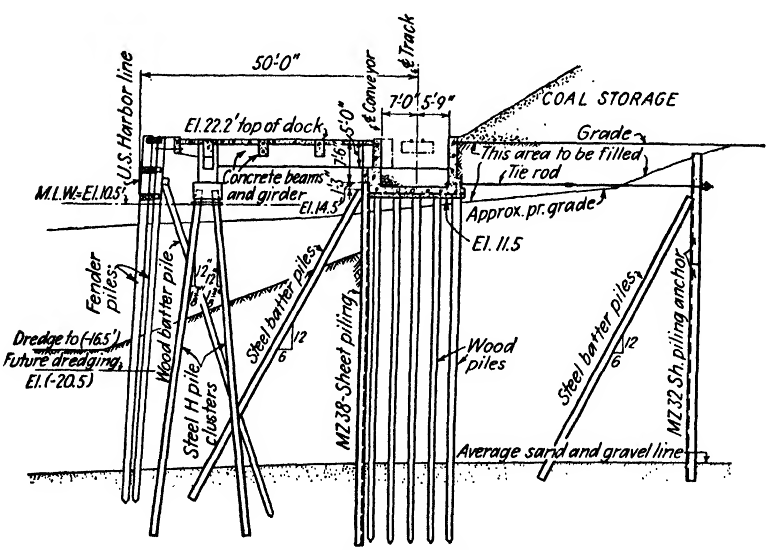


FIG. 10-41. Bulkhead construction along the edge of the coal-storage area at a power plant. The tracks are for one end of the loading and unloading bridge. (Courtesy of Arthur G. Beaulieu, *Bulkheads: Their Purpose — Design and Construction, 56th Annual Report, The Connecticut Society of Civil Engineers*, 1940.)

was dumped from the shore outward instead of in reasonably horizontal layers. The tie rods were bent down so badly in places that they pulled the bulkhead wall far out of line. One tie rod failed.

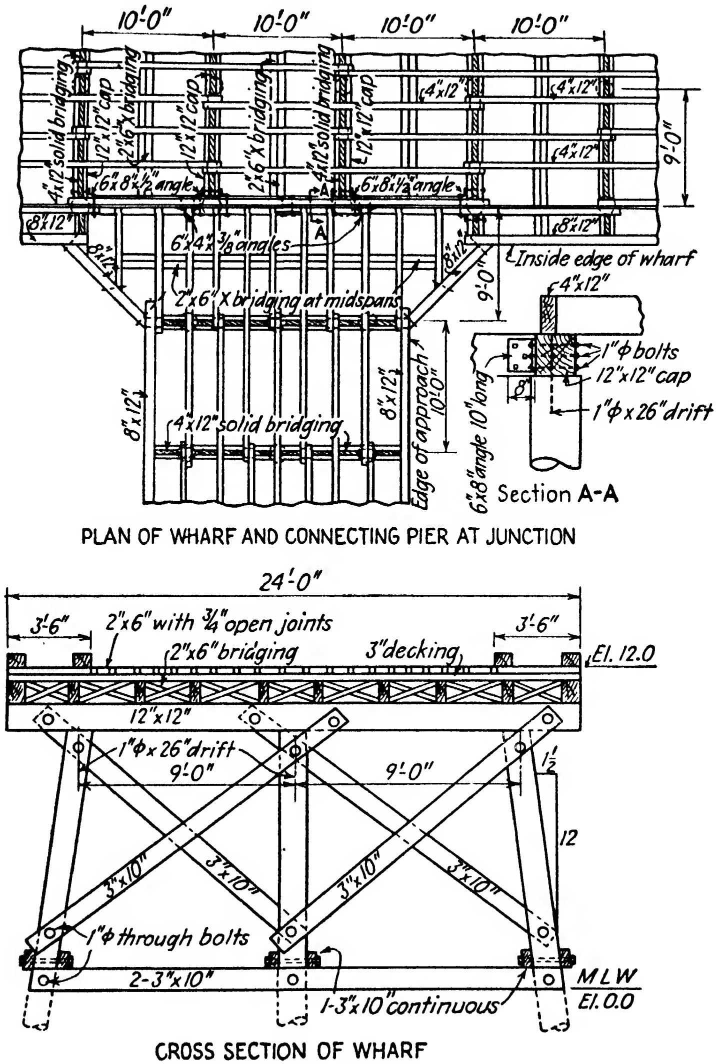


FIG. 10-42. Creosoted timber pier and wharf construction at the Charleston, S.C., Navy Yard. *(Courtesy of Wood Preserving News, January, 1945.)*

Tie rods might be encased in concrete for preservation, but this adds to the weight and the transverse vertical earth and live loads carried by the rods. It is better to use large-sized rods to allow for corrosion, and it is advisable to paint them with hot asphalt or some other suitable coating or to wrap them in membrane waterproofing.

The bulkhead shown in Fig. 10-41 is subjected to heavy loads. The coal-storage pile might be 50 ft. high. The tracks shown are to support one end of a gantry crane having a 300-ft. main span and a 120-ft. cantilever over the water to unload colliers having a draft of 30 ft. Hurricane winds might cause a load of 600 kips on the leeward wheels of the gantry, also a horizontal thrust of 167 kips. These forces might be applied along the track. Notice the combination of steel and wooden piles, the trench for the tracks, and the braced sheet-pile anchorage.

10-17. Evaluation of pile groups. As stated previously, tests of piles that bear on firm soils generally yield reliable results for use in determining the safe load per pile when used in large groups, unless they are underlain by weak cohesive soils. The tough problem is the selection of the presumptive bearing power when the piles are in large groups in and above plastic cohesive materials. Another problem related to this is estimating the probable settlement of the foundation, the determination of whether or not this settlement will be harmful, and the making of a decision regarding what should be done about it.

Some engineers refer to the efficiency of the piles in a group. This term is used to denote the ratio of the safe average bearing value of a pile in the group to that of an identical individual test pile driven to the same depth in the same soil. This is another way of allowing for the fact that the average bearing capacity of one pile in a group of friction piles, as limited by the strength or settlement of the soil, is usually less than that of an isolated

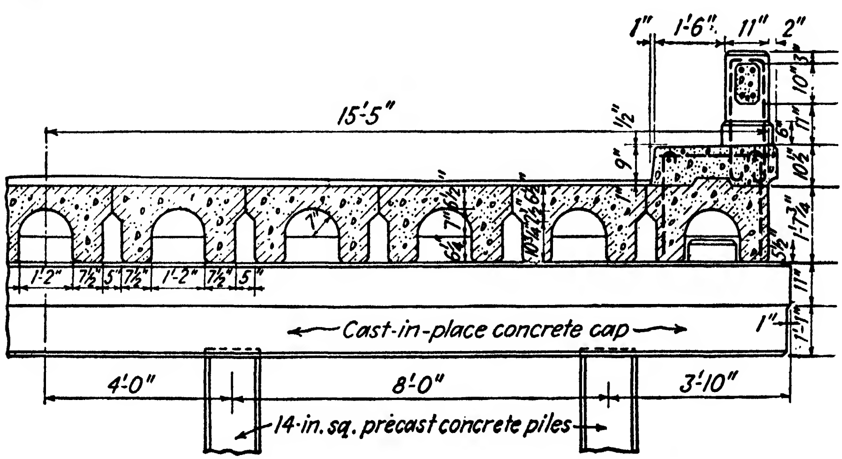


FIG. 10-43. An example of precast concrete bridge construction. Intermediate bents consist of four concrete piles with a capacity of 25 tons each. The roadway and sidewalk supports are precast sectional slabs. The curb, sidewalk, and handrail are cast in place. (*Courtesy of E. S. Fraser, Precast Concrete Slab Bridges in Florida, Reinforced Concrete, Portland Cement Association*)

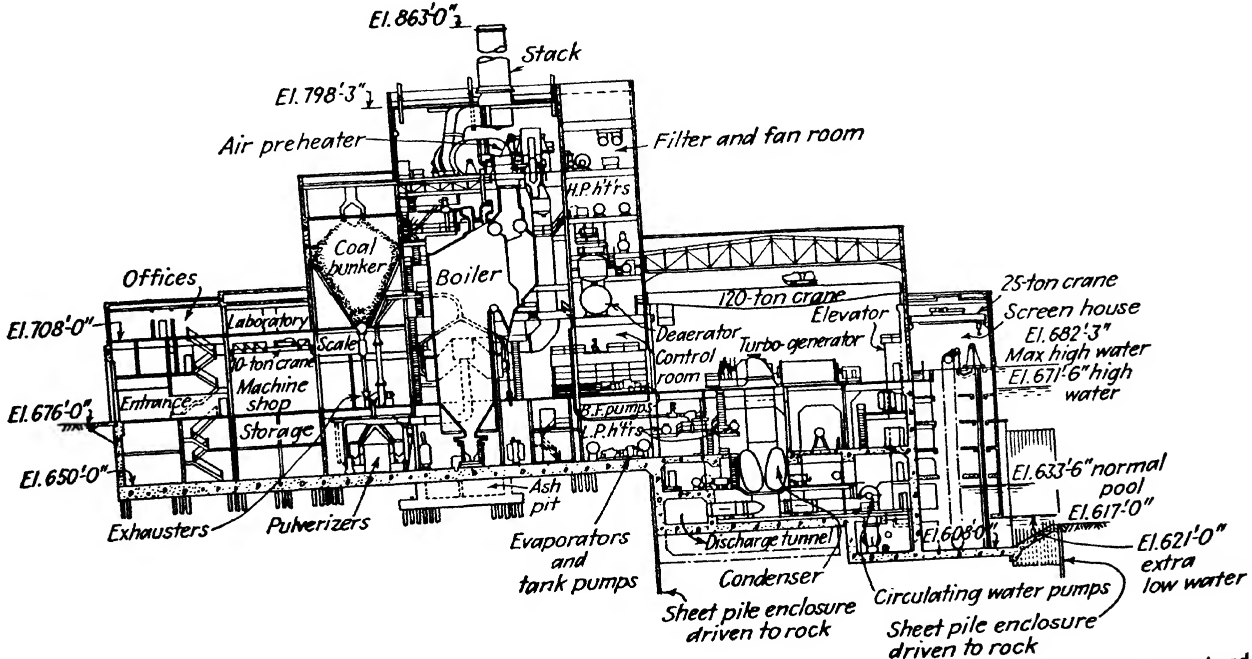


FIG. 10-44. Longitudinal section through a power plant. Note that part of the structure is supported by piles and part rests directly upon sand and gravel. Concrete walls are used below flood level. (See H. A. Kammer, Modern Steam Electric Power Station Built on Riverside Site, *Engineering News-Record*, Oct. 31, 1946.)

test pile. It is obvious that attempts to determine this efficiency are, and have been, unsatisfactory because local conditions differ so much at various sites, and they are not dependent upon the number and type of the piles. It is better to analyze a problem upon the basis of the estimated bearing value and stiffness of the soil, looking upon the piles primarily as a device for getting the load "into" the earth.

Much remains to be learned about the predetermination of the number and spacing of friction piles needed to support a given load under given conditions. Theoretically, for a particular required bearing area just below

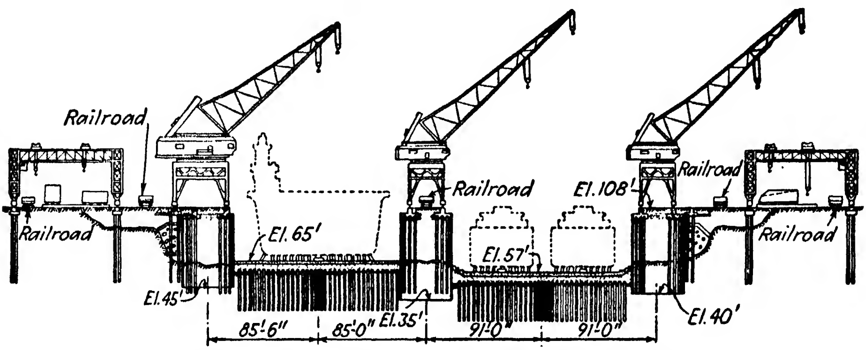


FIG. 10-45. Section through submerged shipways built for the Navy Bureau of Ships. Cellular cofferdams later became a permanent part of the walls. Wooden piles are used under the shipway area and to support the gantries above the cells. (See *Adolph J. Ackerman and C. B. Jansen, Submerged Shipways with Steel Sheeting Walls, Civil Engineering, July,* 1943)

the piles, the greatest economy will result when the number of friction piles and their spacing are such that the frictional resistance along the surface of each pile is almost completely utilized in the transfer of the pressure to this soil. This tends to influence the use of wide spacing and long piles under many conditions. This, however, may cause the use of wider and more expensive footings, and of more costly piles and equipment. The engineer who is planning a specific foundation dares not let his desire for economy carry him so far that his structure will be endangered. On the other hand, it is impracticable to make load tests of large groups of piles at various spacings because of the great cost and delay. Furthermore, the results of small-scale model tests are likely to be delusive. Service records of actual foundations are almost the only way to increase our knowledge sufficiently to avoid danger on the one hand and too costly conservatism in design on the other.

The following procedures are given as suggestions that may be used in addition to loading tests in obtaining quantitative data as a basis for judgment in the selection of the allowable load per pile for purposes of design. These are based upon the idea that the supporting power of the earth below the tips of the piles is the critical part of the design. This includes the avoidance of excessive consolidation under load as well as the ability to hold up that load. Naturally, the results will not be accurate, but they may be helpful in comparing various possible solutions for a problem involving groups of piles in cohesive soils.

1. Select a type of pile and a depth of penetration that seem to be suitable for the conditions.

2. Estimate the static frictional resistance that may be developed along the embedded surface of the pile, using the area of the surface of the pile and the unit frictional resistances taken from Table 9-1. If the result of a pull-out test is available, that will be even more useful.

3. Deduct the weight of the pile, neglecting buoyancy, because the weight of the pile itself is not "useful" load but requires some of the friction to hold it up. Wooden piles have the advantage of lightness.

4. Having the result of item 2 minus 3, discount somewhat the total load per pile because of the group effect, if this seems to be necessary, although the values given in Table 9-1 already have allowed a little for the effect of small groups. Compare with Table 10-1. From these select a trial safe load per pile. Determine accordingly a tentative number and arrangement for the piles, and make a rough sketch of the scheme in plan and vertical section.

5. From the top of the earth that has reasonable supporting value, draw assumed distribution lines like AC and BD, Fig. 10-46(*b*). These intersect the plane CD at the tips of the piles so as to delineate the area $CC'O'OD'DNN'$ in Sketch (*d*).

6. Assume that the pressure diagram on the area outlined in Fig. 10-46(*d*) is shown in cross section in (*c*) as $GCDH$. This is flat under the piles and decreasing in intensity from area $EE'F'F$ to the perimeter. It is probable that the actual distribution of pressure will be more like the dotted line PQ in (*c*), but the difference will be neglected. However, for the purposes of computation, it is satisfactory to assume that the diagram of intensity is represented by $RSTU$ in Sketch (*c*) and that $SS'T'T$ in (*d*) represents the area over which it is theoretically applied. Notice that RG is $\frac{1}{2}CE$, etc.

To avoid confusion, it is desirable to use one method uniformly. The rectangular diagram is so much more simple that it will be assumed. The batter lines therefore will be considered to be those that determine the supposed edges of this diagram, SR and TU in Fig. 10-46(*c*). These are shown as AS and BT in Fig. 10-46(*b*) in contrast to lines AC and BD originally assumed. If the soil is very soft clay or weak silt, assume a batter of 10:1 to 12:1; if moderately soft clay or compacted silt, 6:1 to 8:1; if medium clay or stronger, 4:1 to 8:1. The choice of one of these values of batter will be affected largely by the cohesion and angle of friction of the soils around the piles—their ability to resist vertical shearing action and to spread the load. These values are based upon the assumption that the soil is fairly uniform. If its strength and stiffness increase with the depth, the center of gravity of the frictional resistance will be lower and there will be more tendency to concentrate the load nearer to the tips.

7. Compute the volume V of the pressure diagram. The volume of this pressure diagram equals the superimposed load W. This need not include

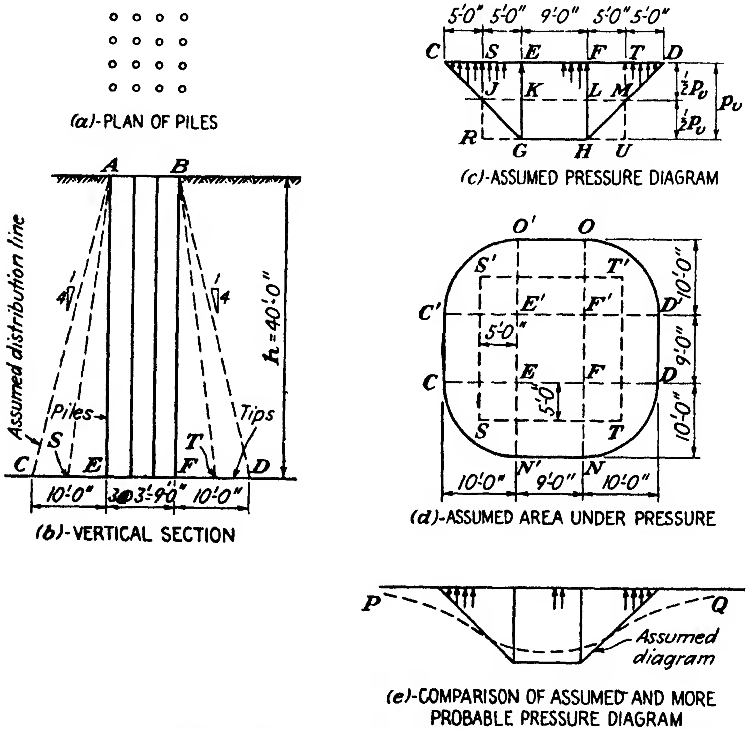


FIG. 10-46. Scheme for estimating pressure on soil at tips of piles of a group in cohesive soils.

the weight of the piles themselves except when this item is significant, because they can be considered as replacing equivalent earth even though they may compact the soil somewhat during the driving. The weight of the overburden is not considered to increase the pressure applied to the soil because it is already there.

8. The computed maximum soil pressure p_v is then

$$p_v = \frac{\text{total load on foundation}}{\text{area}} = \frac{W}{A}$$

9. Compare p_v with the load p that is considered to be safe for the soil at the tips of the piles as determined by tests or otherwise. If p_v exceeds p, reduce the assumed safe load per pile accordingly; if p_v is less than p,

the safe load per pile might be increased if settlement, the frictional resistance, and load tests of individual piles seem to warrant it.

10. Revise the layout to utilize the piles to maximum practicable advantage if the test load per pile is considerably greater than the computed value as a member of the group. This generally means that the use of wider spacing is economical except as limited by practicable sizes of footings or caps.

To illustrate this last paragraph in reverse, assume that the soil below a group of 16 piles at 4-ft. spacing seemingly overloads the soil below them. The situation will not be improved by driving more piles between the others to obtain a group of 49 piles at 2-ft. spacing. Spreading the spacing to 5 ft. would help slightly but might make an unduly large footing. A better remedy would be the deepening of the penetration to obtain more spreading or to reach firmer ground.

The next problem is the invention of a procedure for estimating the settlement of a pile foundation. The best that can be expected is a determination of the general scale of the magnitude of the future settlement. Whether it will be 2 or 2½ in. means little. Whether it will be 2 in., 6 in., or 2 ft. means much.

The following procedures are suggested for making an estimate of the settlement of a group of friction piles supported by plastic cohesive soils:

1. By laboratory tests for compaction under load, obtain data giving the relation of pressure vs. consolidation, and plot curves giving these data. Do this for each 10 ft. of depth of soil around the piles, and for considerable distance below them, when the properties of the soil vary considerably.

2. Compute the pressure diagram as outlined previously.

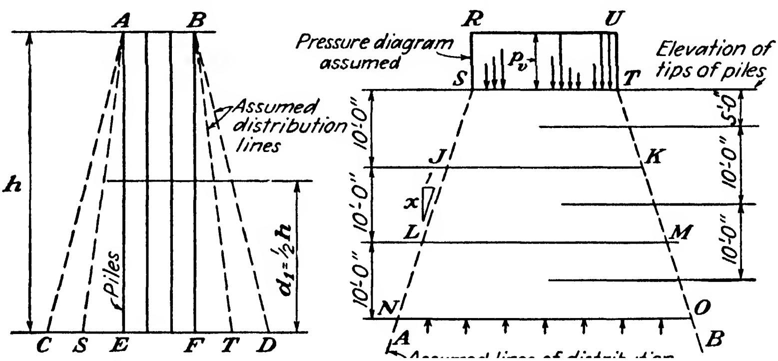
3. Assume the intensity p_v to act upon the soil around the piles for the lower half of the embedded depth, h, of Fig. 10-47(a). Find the computed settlement for the depth d_1, or the individual settlements for each of the 10-ft. layers in the distance d_1 if the soils vary.

4. Assume the intensity p_v and the pressure diagram computed in item 2 to act over the area directly under the group of piles. This is pictured as the width ST at the top in Fig. 10-47(b). Assume a theoretical spreading of this load at some batter $x:1$, where x is consistent with the properties of the soil below the piles. An assumed magnitude for x may be 3:1 for medium clay and well-compacted silt; 2:1 for firmer plastic soils and fine sand. Then compute the resultant pressure p'_v for similar pressure diagrams at the centers of the 10-ft. layers below the piles, including as many layers as the magnitude of p'_v and the soil conditions seem to warrant. Notice that pressures below ST in (b) are to be computed at the centers of the assumed layers.

5. Compute the compaction of each of these layers, then add all the computed results of items 3 and 4 to obtain the estimated settlement.

6. Mentally back off and look at the whole problem in proper perspective. Are these results seemingly reasonable? Do they show any danger signals? Test the results again with a different (greater) assumed spreading batter. Estimate the settlement by any other applicable method. If the results persistently show computed settlements that will be harmful to the structure, devise some way to remedy the situation, even to modifying the superstructure, changing the substructure scheme, or putting the structure somewhere else.

Paragraph 3 needs explanation. A tapered pile has more surface area near its butt than near its tip. Therefore, it would seem that the pressure



(a)-VERTICAL SECTION THROUGH PILES

(b)-ASSUMED DISTRIBUTION OF PRESSURE THROUGH SOIL BELOW PILES

FIG. 10-47. Scheme for estimating settlement of a group of piles in and above cohesive soils.

transmitted to an annular area of soil near the butt, pictured by BC of Fig. 10-48, would have a greater increment of compression per square foot than would a corresponding ring LM near the bottom, assuming the properties of the soil to be the same at both places. The pressure at BC will tend to decrease in intensity as it is transmitted downward over the increasing assumed area to HM. Nevertheless, the soil below BC cannot resist compression without deformation. If A and M were to remain stationary, since the pile is relatively incompressible, any shortening of the ground from BC to HM would have to be taken up by cracks or tension, both of which are inconsistent with what must occur. As the lower soils compress, the pile must penetrate farther into them in order to have compressive stress in the ground around it. Since AH is assumed to be a limit to the lateral spreading of significant compressive stresses in the ground, it would seem that the forces on ring BC would spread over the whole of ring HM; EO over JM; PN over KM; and LM over itself. Therefore, the intensity on HM will probably decrease from M outward to, and perhaps beyond, H. The lower strata are also often more compact and resistant than the higher ones, and the driving may tend to remold the clay somewhat. Therefore, with so many uncertain elements, it seems to be sufficient to assume a broadly interpreted compacting effect equivalent to the computed maximum pressure p_v acting through a depth equal to the lower half of the embedment.

There remains much to be done in correlating the results of experience with any proposed mathematical procedure for predicting the safe loads and settlements of groups of friction piles. Numerical values given herein for the assumed batter, and the suggested procedures in their entirety, are

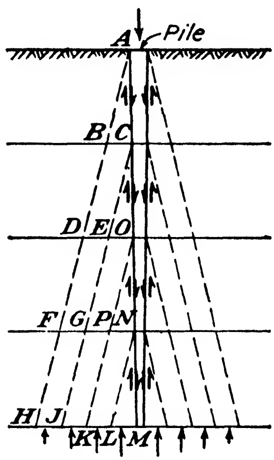


FIG. 10-48. Assumed action of soil around a friction pile.

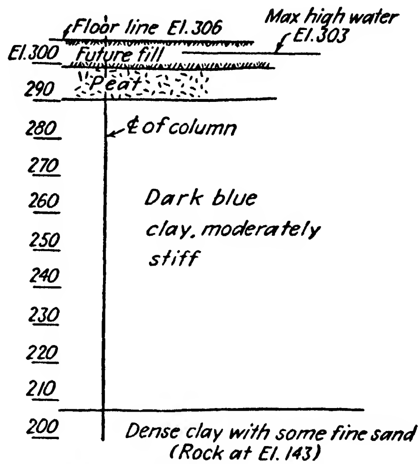


FIG. 10-49. Soil profile at a proposed industrial plant.

proposals. Their accuracy cannot be expected to be perfect. Even past performances seem to have been exceedingly variable.

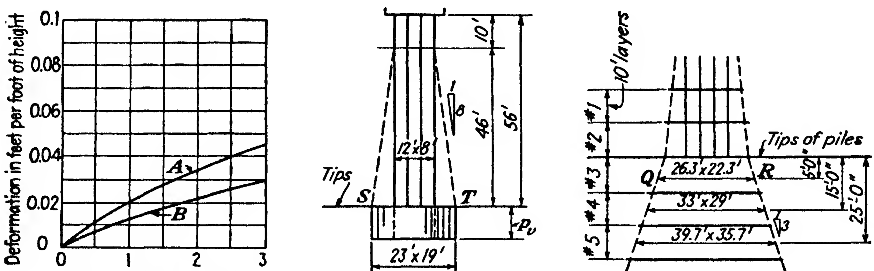
Now, test all these procedures by application to real situations.

Example 1. The soil pictured in Fig. 10-49 is at the location of footings that are to support heavy columns 30 ft. c.c. in an industrial plant. One total estimated load, including the footing, is 240 kips *DL* and 320 kips *LL*. Select the type and length of pile to be used, plan the pile layout, and estimate the settlement. The clay is fairly uniform. Assume the data in Fig. 10-50(a) for its compaction, using curve A. Assume further that the engineer in charge believes that the deeper clay should not support a load of more than 2 k.s.f. because of settlement.

Friction piles are necessary. The peat should be removed before the fill for the floor is placed in order to avoid large future settlement of the floor and machinery. This, since drainage of the fill is to be installed, will permit a low watertable during a dry season, making plain wooden piles undesirable. Steel H's and pipe piles are not suitable because their great strength cannot be developed by the friction unless they are excessively long. Creosoted piles or thin-shelled cast-in-place piles of Raymond or Monotube type seem to be advantageous. The Raymond step-taper pile shown in Fig. 9-10 will be selected, with the cutoff at El. 301. Assume 56-ft. lengths as suitable; great length does not seem to be necessary. The tips will be at El. 245.

With the data in Fig. 9-10 giving surface areas, and a unit friction of 300 p.s.f. from Table 9-1, the estimated frictional resistance of a single pile is

$$F = 166 \times 0.3 = 50 \text{ kips}$$



(a)-SOME ASSUMED CURVES SHOWING COMPRESSIVE DEFORMATION OF SOILS VS. COMPRESSIVE LOAD

(b)-SECTION THROUGH PILES

(c)-LAYERS FOR COMPUTING SETTLEMENT

FIG. 10-50. Data for an estimate of settlement.

The fill is not relied upon to help support the pile. The buoyant weight of the pile is approximately 4 kips. Therefore, the net supporting friction $F' = 46$ kips. Because of the group effect, call the bearing value 40 kips per pile. This requires

$$N = \frac{240 + 320}{40} = 14 \text{ piles (assume 16)}$$

Try a square group of 16 piles at 4 ft. c.c.

Now test for the pressure on the soil. To be very conservative, assume distribution at an 8:1 batter from the top of the clay, El. 290. Then

$$W = 560 \text{ kips} = p_v(12 + 11)(12 + 11)$$
$$p_v = 1.1 \text{ k.s.f.}$$

This is less than the 2 k.s.f. allowed, so that the supporting power of the soil is not critical.

It is now desirable to see whether a higher load per pile can be used, since the soil seems to be strong enough. If the full 46 kips per pile were allowed, the revised number required would be

$$N' = \frac{560}{46} = 12.2 \text{ (call it 12)}$$

For the present, assume these as three rows of four each, all at 4-ft. spacing. The dimensions are shown in Fig. 10-50(*b*). Compute the revised soil pressure

$$p_v = \frac{560}{23 \times 19} = 1.3 \text{ k.s.f.}$$

In estimating the long-term settlement of the piles, the live load is neglected because crane loads constitute most of it, and these are temporary. The effect of the added fill over the whole area is not included in this particular problem, but it should not be overlooked.

The computations are as follows, referring to Figs. 10-50(*a*) and (*c*) and using a batter of 3:1:

$$p_v \text{ for dead load } = 1.3 \times \frac{240}{560} = 0.6 \text{ k.s.f. (approx)}$$

Above tips, using curve *A*:

$$\Delta_1 = 10p_v \times 0.013 = 0.13 \text{ ft.}$$
$$\Delta_2 = \Delta_1 = 0.13 \text{ ft.}$$

Below tips: Compute the pressure at *QR*, the middle of layer No 3.

$$p_3 = \frac{240}{26.3 \times 22.3} = 0.4 \text{ k.s.f.}$$
$$\Delta_3 = 10 \times 0.01 = 0.10 \text{ ft.}$$

Similarly,

$$p_4 = \frac{240}{33 \times 29} = 0.25 \text{ k.s.f.}$$
$$\Delta_4 = 10 \times 0.007 = 0.07 \text{ ft.}$$
$$p_5 = \frac{240}{39.7 \times 35.7} = 0.17 \text{ k.s.f.}$$
$$\Delta_5 = 10 \times 0.004 = 0.04 \text{ ft.}$$

The total computed settlement thus far—the sum of the five 10-ft. layers used—is 0.47 ft. It is obvious that the unit pressure on the still lower strata will decrease to insignificance. Therefore, it is reasonable to conclude that, according to these computations, the long-term settlement of this footing will not exceed 6 or 7 in. Is this objectionable? It certainly would be if one column alone were to settle this much more than its neighbors. If all settlements could be kept the same, no serious harm would result. This, however, will be difficult since the column loads vary considerably. Besides this, the live loads are large and include some vibrations. The footings should not seem to sink into the surrounding floor. Furthermore, overlapping of the effects of loads on adjacent columns has been disregarded. More conservatism seems to be desirable. Assuming the clay to be as compressible as Fig. 10-50(*a*) and these computations indicate, settlement of footings and general settlement of the area under the weight of the new fill will be troublesome. Of course, the assumption of a flatter distribution in Fig. 10-50(*b*) would yield a smaller computed settlement, but this would not change the general character of the result.

Spreading the piles farther will gain little advantage. The addition of more piles also gains little compared to the extra cost. Driving the 12 piles 8 to 16 ft. deeper is more advantageous. If this is done, what effect will it have upon the computed settlement? It looks as though the result would still be unsatisfactory unless considerably longer piles could be used to reach the dense clay and sand farther down. This might be a good place to use composite piles with wooden piles at the bottom and poured-in-place concrete for the upper portion.

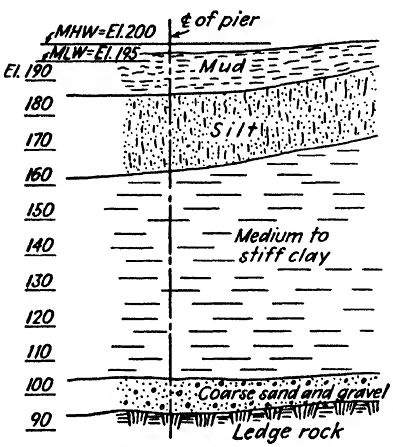


FIG. 10-51. Soil profile at a proposed bridge pier.

Example 2. Figure 10-51 shows the soil profile at a small bridge pier consisting of two concrete shafts 6 by 8 ft. in section and 30 ft. c.c. at El. 200. The total load at El. 200 for this study is 1,650 kips *DL* and 450 kips *LL*. It is thought to be advisable to use piles under two separate footings or under a combined footing. Determine the type, length, and arrangement of piles. Estimate the settlement and the soil pressure below the piles.

Here, 100-ft. H piles or pipe piles could be used directly down to rock. Precast concrete piles to firm end bearing would be rather long and heavy, but they could be used if the base of the pier were set at approximately El. 180, which might very well be desirable. Wooden piles into the soft clay could be used also, but they would have to be too long if they were to reach the sand and gravel. Poured-in-place thin-shell concrete piles could be used in the clay, but they too would be rather long if they were to reach the sand. Button-bottom piles could be driven to the sand, and a thin shell could be used on the main shaft to keep out the silt. The choice is between long strong end-bearing piles and shorter weaker friction piles.

Before making a decision, test 50-ft. wooden piles with their butts at El. 185. These are to be driven inside a cofferdam that permits the excavation of the mud and the upper portion of the silt. Assume that these piles are 8 in. at the tip and 20 in. at the butt. Assume 20 ft. in silt and 30 ft. in clay. From Table 9-1, assume 100 p.s.f. for skin friction in silt; 400 p.s.f. in clay. Use the area of the surface of a cylinder having the mean diameter of each of these portions of pile instead of the tapered sections.

F_s for silt $= 0.1 \times 20 \times \pi \times 1.47$	....	9.3
F_c for clay $= 0.4 \times 30 \times \pi \times 0.97$	....	36.5
Total F	....	45.8 kips, or 46 kips each

Because of their light weight, no deduction will be made for this from F. However, because of the group effect, and assuming the clay to be safe for only 2 k.s.f. because of settlement, call the safe load per pile 40 kips.

To the loads given, add a guessed net weight of 300 kips for the pier shafts and footing below El. 200. The number of piles required would then be

$$N = \frac{2,400}{40} = 60$$

This seems to require a combined footing under both shafts. Therefore, try four rows of 15 piles each. With four rows at 3 ft. 6 in. spacing, and with the piles at 3 ft. c.c. in each row, this would make a group 10.5 ft. wide and 42 ft. long.

Using an average distribution batter of 10:1 in the 20 ft. of silt and 6:1 in the 30 ft. of clay, the estimated pressure at El. 135 is

$$p_v = \frac{2,400}{(10.5 + 14)(42 + 14)} = 1.75 \text{ k.s.f.}$$

This appears to be fairly large, but the settlement should be investigated before a decision is made.

To make this estimate of settlement, assume p_v to affect only 15 ft. of clay above the tips, because the silt does not resist much anyway. Assume a distribution of 2:1 for the clay below the tips, El. 135. Let curve B of Fig. 10-50(a) represent the load-deformation relationship for this clay. It is so nearly a straight line that it is assumed to be one, and the 30 ft. of clay from El. 135 to El. 105 will be used as one single 30-ft. layer. The area of the assumed pressure diagram at El. 120—the center of the 30-ft. layer—is 71 by 39.5 ft. Use the dead load of 1,950 kips when computing the settlement. Then,

Above tips:

$$p_v = 1.75 \times \frac{1,950}{2,400} = 1.4 \text{ k.s.f.}$$

$$\Delta_1 = 15 \times 0.017 = 0.26 \text{ ft.}$$

Below tips:

$$p_v' = \frac{1,950}{71 \times 39.5} = 0.7 \text{ k.s.f.}$$

$$\Delta_2 = 30 \times 0.009 = 0.27 \text{ ft.}$$

The total computed settlement is 0.53 ft. or 6 to 7 in.

Is this satisfactory? It raises serious questions because greater settlement is not guaranteed by any of these figures. It is not a matter of weakness of the piles but of consolidation of the soil under and around them.

Suppose that end-bearing piles driven to rock are considered now. At 50 tons per pile, the number would be

$$N = \frac{2{,}400}{100} = 24$$

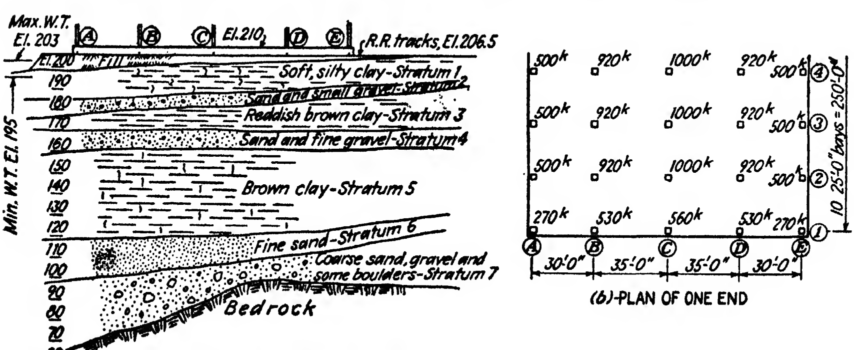
These could be arranged as two groups of three rows each at 4-ft. spacing under each pier shaft, provided the lateral stability will be satisfactory. In this case, there is no question of settlement. Is the cost greater or less than for the wooden piles? If greater, is the security worth it?

If the end-bearing piles cost \$10 per foot in place whereas the wooden ones cost \$4 per foot, the latter would be cheaper, but some additional expense would be entailed in the cost of the larger footing.

The final choice will depend considerably upon the method of construction desired, but it seems advisable to eliminate all question of harmful settlement by using long piles that reach the sand and gravel. Assuming that floating equipment cannot be used without too much dredging, the proper procedure seems to be the construction of the cofferdam from a temporary wooden-pile trestle, the excavation to El. 185 for the base of the pier, then the driving of 16- or 18-in. closed-end pipe piles with the ends capped and welded watertight. These pipe sections could be made of two pieces welded in the field, if one piece is too long to handle. The pier might then have two groups of 12 piles each, with a bottom strut of concrete joining the two sides.

Example 3. Figure 10-52(a) shows part of the soil profile at the site of a fireproof warehouse. The design loads and column spacing are given in (b). Assume that 70 per cent of the loads given represent live loads. Select the type of foundation to be used, and design a typical support for column C3.

The top stratum is too weak to support spread footings. The column loads and spacing are too great for the practical use of a mat foundation.



(a)-SOIL PROFILE AT COLUMN LINE ③

FIG. 10-52. Soil profile at site of proposed fireproof warehouse.

Strata 2 and 3 are better soils but are too far down to reach with footings. Stratum 4 is about 12 to 15 ft. thick and reasonably firm. However, it is underlain by nearly 30 ft. of clay. What should be done? Can piles be supported by stratum 4, or must they be driven clear down to stratum 6?

Before attempting to answer this, estimate the settlement of the brown clay if the piles penetrate to El. 165, just above stratum 4. The average load of the column when spread over the building area is 1.2 k.s.f. For settlement computations assume 50 per cent of the live load as applied for a long time. The average superimposed load of the superstructure is then

$$p = 0.3 \times 1.2 + \frac{0.7}{2} \times 1.2 = 0.78 \text{ k.s.f.}$$

To this add 75 p.s.f. for the weight of the first-floor slab that is supported upon the fill, and add 100 p.s.f. for the live load on the first floor. Assume also that the average fill placed over stratum 1 is 8 ft. at 120 p.c.f. The total superimposed load then is

$$p = 0.780 + 0.075 + 0.100 + 0.960 = 1.92 \text{ k.s.f.}$$

Assume at first that no piles are used and that the load is distributed laterally through the ground at a rate of 2:1. Then the added pressure at El. 135 is, approximately,

$$p_{135} = \frac{1.92 \times 135 \times 255}{(135 + 70)(255 + 70)} = 1.0 \text{ k.s.f.}$$

Assuming curve B of Fig. 10-50(a) to represent the load-consolidation curve of the brown clay, the estimated settlement is

$$\Delta = 30 \times 0.013 = 0.39 \text{ ft.}$$

This appears to be satisfactory if it is fairly uniform over the entire area. Furthermore, nearly one-half of the settlement is caused by the fill. The fill is tapered slightly, but this should not be serious. On the other hand, if the piles penetrate far into stratum 4, they will tend to concentrate the pressure on the top of stratum 5. That is why their tips are to be near El. 170—penetrating slightly into the sand and gravel—in order to secure at least the benefit of end bearing in stratum 4.

Assume that the length of piles below the top of stratum 1 is approximately 30 ft. The minimum watertable is a little too low for plain wooden piles. The great strength of precast concrete piles cannot be developed by the friction alone, but they would have a large surface area and a good end for bearing. Furthermore, they are a good type for driving through the thin but dense crust of stratum 2. Poured-in-place concrete piles without shells do not seem to be desirable, partly because of the danger of penetration of the saturated sand and gravel of stratum 2 when concreting the piles. Metallic shells seem to be desirable if they can be driven through stratum 2 without rupturing. Steel H piles do not seem to be advantageous here. The choice appears to lie between creosoted wooden and cast-in-place concrete piles. Since the surface areas are given in Fig. 9-10, use Raymond standard piles in this study.

The estimated average frictional and end-bearing resistance, using figures from Table 9-1 and Fig. 9-10, is found as follows:

Stratum 1: 250 p.s.f. friction, 14 ft. thick
Stratum 2: 600 p.s.f., 5 ft. thick
Stratum 3: 400 p.s.f., 11 ft. thick
Stratum 4: end bearing

Stratum 1: 0.25×63	=	16 kips
Stratum 2: 0.6×14	=	8 kips
Stratum 3: 0.4×33	=	13 kips
Stratum 4: estimated	=	6 kips
		43 kips (or call it 44)

Deduct 4 kips for the weight of the piles.

The number of piles for footing C3 is then

$$N = \frac{1,000}{40} = 25$$

Use these as a square group at 3-ft. spacing. Have the tops at about El. 204 for interior footings and El. 201 for exterior ones.

There will be a tendency to have larger pressures on stratum 5 near the ends of the piles than in between the footings. However, stratum 4 will spread the loads well enough so that, as shown by the preceding computations, the results should not be harmful. There seems to be no necessity for deeper penetration of the piles. It is also probable that the group effect will not weaken the assumed supporting value of the piles because strata 4 and 5 are good materials. Load tests of the proposed piles should be made to ascertain whether or not the safe load can exceed the 20 tons per pile assumed. This would probably prove to be the case.

PROBLEMS

Some of the data in the Appendix is useful in working out problems. Where additional data are desired, they are to be assumed and the problems are to be worked out accordingly.

10-1. Figure 10-53 shows the logs of some borings that were taken at the site of a proposed steel mill building to be located in the yard of an industrial plant alongside a river near the coast of New Jersey. The building is to house a few heavy but widely spaced machines to be served by 50-ton cranes. Make a preliminary investigation to determine the type of foundation that is best. If piles are used,

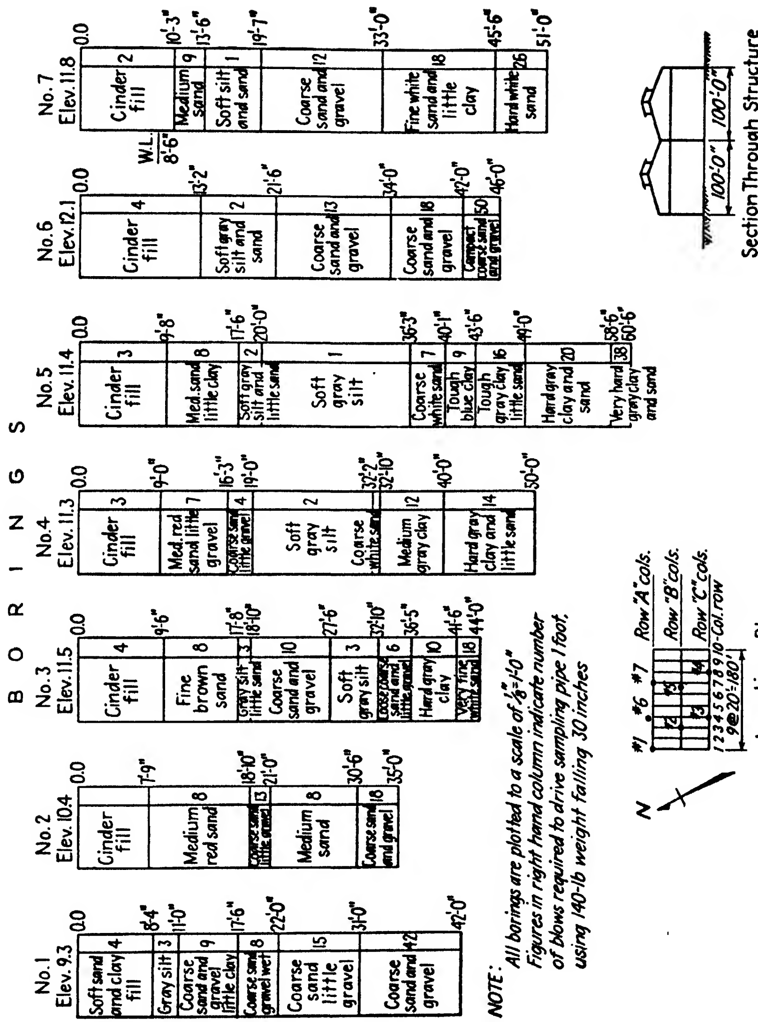
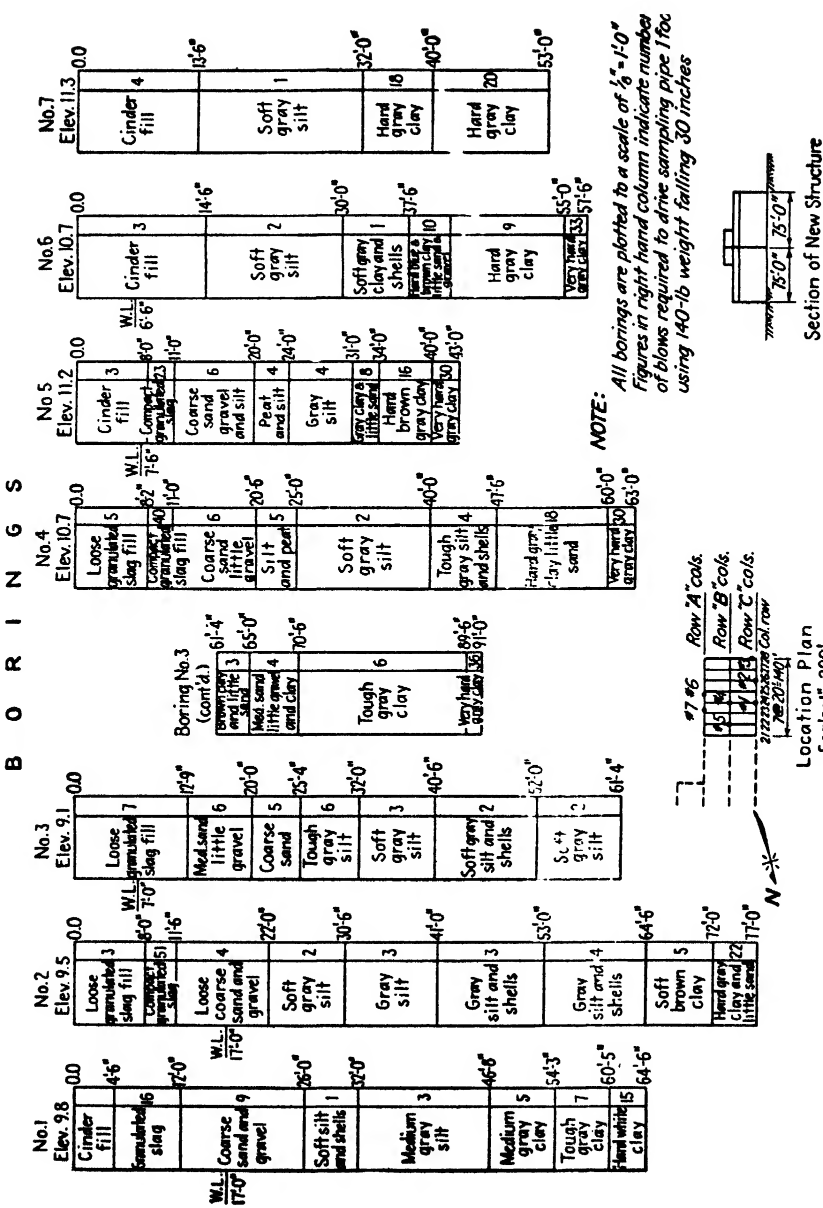


FIG. 10-53. Logs of borings.



NOTE: All borings are plotted to a scale of $\frac{1}{8}$"=1'-0". Figures in right hand column indicate number of blows required to drive sampling pipe 1 foot using 140-lb weight falling 30 inches

FIG. 10-54. Logs of borings.

determine the type of pile, their length, and the arrangements for typical foundations.

The borings are not spaced to give as comprehensive information as one might desire, but they are taken from a real job just as they were presented to the engineer.

Suggestions: Using Table 5 in the Appendix, the dimensions shown in Fig. 10-53, and an over-all dead and live load of 100 p.s.f., approximate the load on an exterior column and that on an interior one.

Now draw soil profiles along the column lines and across the building site through borings 3, 2, and 6; also through 4, 5, and 7. These may show the difficulties of fitting the data from the borings into what seem to be reasonable profiles. The past

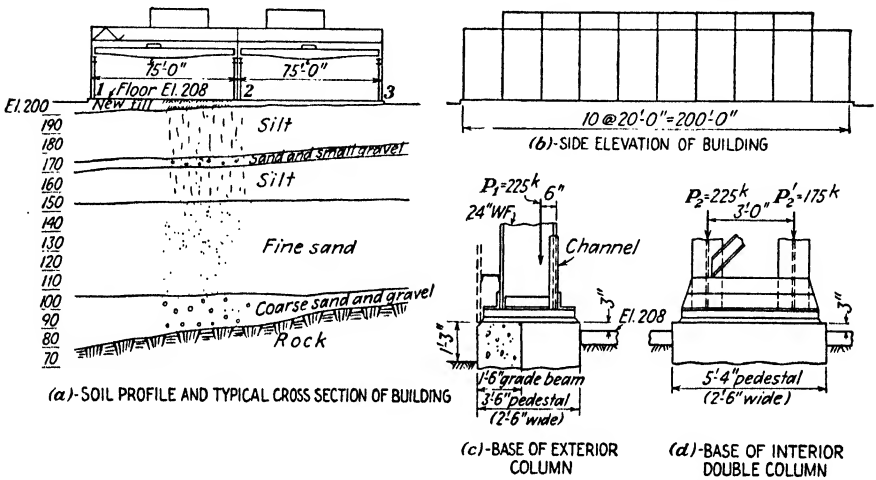


FIG. 10-55. Soil and superstructure data for small industrial plant.

effects of scour, the localization of deposits, the presence of lenses of soil, and the general unpredictability of what lies beyond the borings may become apparent. These are often characteristics to be expected in the lower valleys of rivers, especially in glacial territory.

Some questions to be answered are the following:

1. Is it safe to found these crane columns on the cinder fill?
2. Is it safe and practicable to use spread footings on the uppermost stratum of sand?
3. Should piles be used to reach the strata below the silt?
4. Can piles be used safely for some of the bases and spread footings on soil for the others when cranes are to be supported?
5. What type of pile should be used, and what might the length be?

10-2. In Fig. 10-54 are shown the logs of some borings that were taken at the site of a proposed extension of an existing shop at an industrial plant at the mouth of a river along the Eastern seaboard. The existing structure at the north is founded upon 40-year-old spread footings. The structure is to be a steel-frame mill building with a 30-ft. clearance under the flat and monitored roofs. The walls are to be 8-in. brick. There will be a 25-ton crane between column rows A and B and a 10-

ton crane between B and C. Approximate the column loads and plan the general character and details of the foundations.

10-3. Figure 10-55 shows the soil profile and other data pertaining to a proposed industrial plant. Typical details at the bases of the columns are given and so are

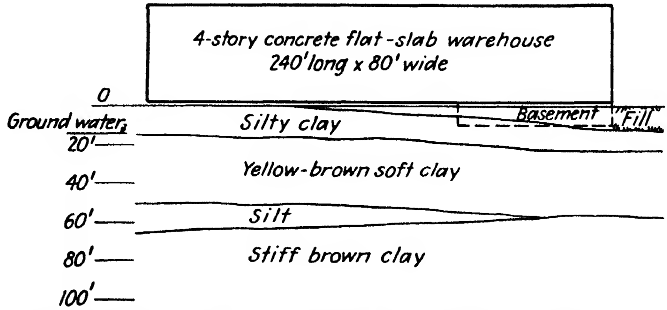


FIG. 10-56. Problem in planning foundation for a warehouse at a tidewater terminal.

the loads. Select the type of pile to be used to support these columns. Design and detail the complete supports for the two bases shown.

10-4. The soil profile shown in Fig. 10-56 pictures the conditions at the site of a warehouse. A basement is at one end only, the remainder of the first floor being directly on the ground. Plan the foundations for the structure, and estimate the probable settlement.

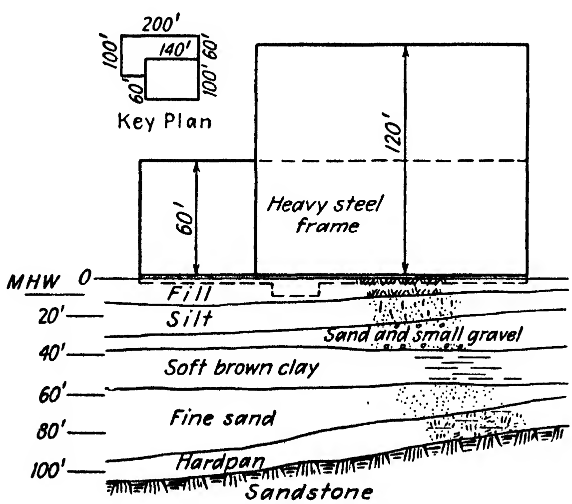


FIG. 10-57. Problem in planning foundation for a heavy steam power plant.

Suggestions: Assume that the columns are 20 ft. c.c. both ways. Assume that the dead load on the interior columns in the basement is 220 kips; elsewhere, 175 kips. The exterior columns have 140 kips dead load at the first floor. Assume that the unit live load for design is 30 p.s.f. on the roof, 300 p.s.f. on all main floors, and 200 p.s.f. in the basement.

10-5. Plan the foundations for the power plant outlined in Fig. 10-57, if it is to be on the soils shown.

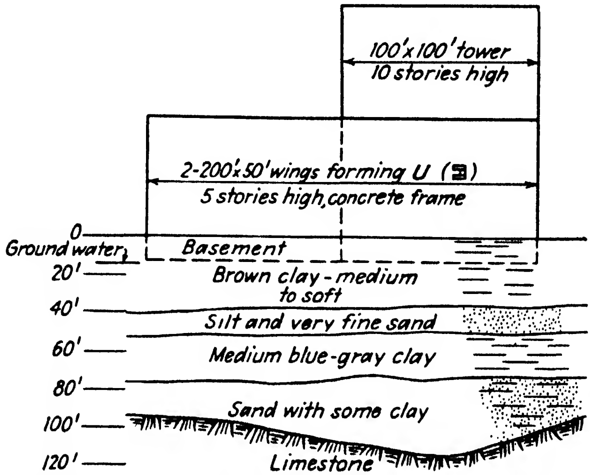


FIG. 10-58. Problem in planning foundation for a large apartment building.

Suggestions: Notice that one portion of the structure is much higher than the rest, and much heavier. Assume that the 200-ft. low portion has a turbine aisle 60 ft. wide and 200 ft. long with an overhead crane and heavy turbine generators. The

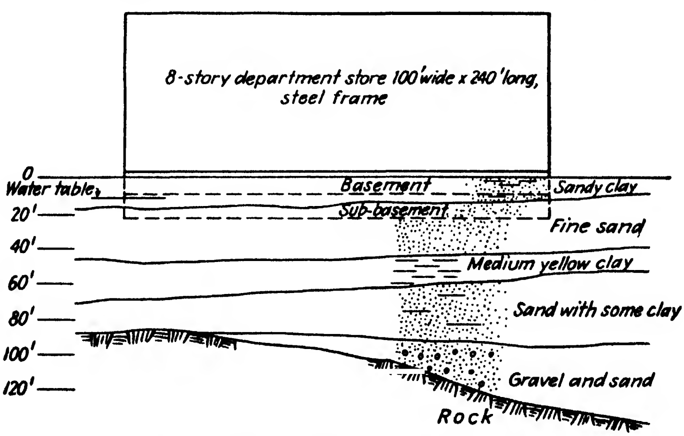


FIG. 10-59. Problem in planning foundation of a large department store.

center of the high portion (boiler house) has two rows of columns spaced as follows: 20, 40, 20, 40, 20 ft. All other columns, interior and exterior, are 20 ft. c.c.

Assume that the maximum column loads in the boiler house are 1,400 kips, that many columns support 400 to 600 kips, and that the smallest loads are 150 kips.

Consider the character of the structure. It is steel frame with concrete floors and brick walls; and it holds very costly heavy equipment.

10-6. Figure 10-58 pictures in outline a large U-shaped apartment house, and it shows a typical portion of the soil profile. It has a basement throughout. Plan the foundations of the structure, and estimate the settlement.

Suggestions: Notice the central tower, twice as high as the two wings. Assume that the columns average 20 ft. c.c. The walls are brick. Assume that the dead load averages 100 p.s.f. for each floor and the roof, including partitions; that the exterior walls average 75 p.s.f.; and that the average live load is 60 p.s.f.

Is this structure suitable for a mat when the loads vary so greatly? Is a floating foundation suitable? Are spread footings safe? If piles are used, what type and length are suitable?

10-7. In Fig. 10-59 is pictured the soil profile at the site of a large store. The structure has a basement and subbasement. Plan the foundations, and estimate the settlement.

Suggestions: Assume that the columns are, in general, 25 ft. c.c. across the building and 20 ft. c.c. along its length. Assume the following loads: dead load of all floors, roofs, and basements, 120 p.s.f.; walls above ground, 75 p.s.f.; live loads, 150 p.s.f.

Is this a good situation for a floating foundation? Are spread footings and walls safe? Will the medium yellow clay be dangerous?

11

COFFERDAMS AND OTHER AIDS FOR OPEN EXCAVATIONS

11-1. Introduction. It is the purpose of this chapter to describe various temporary means used to support the sides of open excavations that are made for the purpose of building substructures. The main discussion is about the features that are essential for the proper planning of these devices prior to construction. As usual, the problems are special ones, each dealing with a particular foundation, built on or in specific soils, and under special pertinent conditions.

The equipment and facilities used for making these excavations are adaptable to a wide range of detail variations. It is desirable to know what means of support are available, what are their possibilities and limitations, and how they can be used to accomplish specific purposes.

Some of the reasons for the need of such temporary supports of the ground are the following:

1. Loose granular soils may have such a flat angle of repose that, in order to make the desired excavation, an unduly large amount of earth would have to be removed unless supports are provided, thus causing an uneconomical procedure.

2. Cohesive soils may stand on steep slopes temporarily, but protection is needed to ensure the avoidance of a sudden cave-in and accident.

3. Slumping of the adjacent earth may undermine adjacent structures, such as buildings, walls, and pavements.

4. The removal of soil that provides lateral support for adjacent ground loaded by structures may cause settlement and cracking of these structures even though a real cave-in does not occur.

5. When work is done below ground-water level, or below river, lake, or ocean level, it is often necessary to surround the excavation with a temporary structure that will hold out the adjacent soils but will also enable the excavation to be unwatered.

6. When there is danger of flooding, a protecting structure may be needed to keep water out of an excavation.

11-2. Shoring. Shoring denotes temporary structures of wood or steel that, in general, are installed as excavation proceeds. It is used to keep the sides of the excavations from caving in. Ordinarily it is used in connection with both large and small but shallow excavations, or for small but moderately deep ones. For example, the trench for a sewer in a street, if not shored, might cause damage to the pavement and so stop traffic; or the sides of a deep pit, if not protected, might cave in and injure workmen. This has happened all too frequently.

A few arrangements of shoring are pictured in Fig. 11-1. The following comments refer to the respectively numbered sketches:

(*a*) This shows wooden shoring installed in a trench. A portion of the trench is dug, and its sides are trimmed to make them reasonably even. The shoring consists of boards or planks *a* that are set horizontally as panels or as single pieces. Vertical beams *b* are added with struts *c* to brace them. The excavation is then deepened, and another piece or tier of shoring is added. This is repeated until the proper depth is reached. This applies to excavation done by hand. If a trencher is used and if the material stands on a vertical slope temporarily, the shoring may be erected after the excavation if the trench is wide enough. Sometimes the shoring need not be extended to the bottom of a trench but can be used only to keep the top from caving in. Again, if the upper material is clay and the lower stratum is fine sand, only the latter need be shored to keep it from running in.

(*b*) This sketch shows wooden members *d* that are erected outside of wales *e* and struts *g* after the upper portion of the trench is excavated. As the digging progresses downward, the vertical pieces are driven farther. Additional wales may have to be added with separators *f*. Posts of some sort are generally needed to hold up the bracing system.

(*c*) The sides of a trench are seldom excavated evenly. The struts *c* and *g* of the preceding sketches bracing the two sides are therefore of variable lengths unless some adjustment is provided. Furthermore, the sheathing should be pushed firmly against the earth if it is to prevent slight slippage of the soil. Nailing the struts to wales or studs would provide some adjustment but not much lateral pressure. A better arrangement is the provision of struts *h* that are somewhat too long. These can be placed diagonally with toe nailing or a cleat at one end. The other end may then be driven along the wale to wedge the sides apart, and then held by nailing cleat *i* to the wale. Again, wedges *j* and shims may be used for the same purpose, and partly nailed to keep them in place. These braces may be used in vertical as well as horizontal planes. Since braces interfere with excavation, the side members should be strong so that the number of braces can be minimized.

(*d*) It is usually difficult to erect shoring directly under an upper portion, as for Sketch (*a*). If the excavation is deep, the sheathing *d* in (*b*) will be too long or difficult to drive. Therefore, a telescoped arrangement as shown diagrammatically here may be advantageous. The first set is made as deep as practicable, then the excavation is deepened, and a second set is installed inside the first, etc.

(*e*) The shoring of pits requires proper details at the corners. Rabbeted ends for pieces *m* may seem to be ideal, but an excavation can seldom be made so that they fit properly, can be erected easily, and are wedged sufficiently. Members *n* can be long; parts *o*, short. Wedges can be inserted

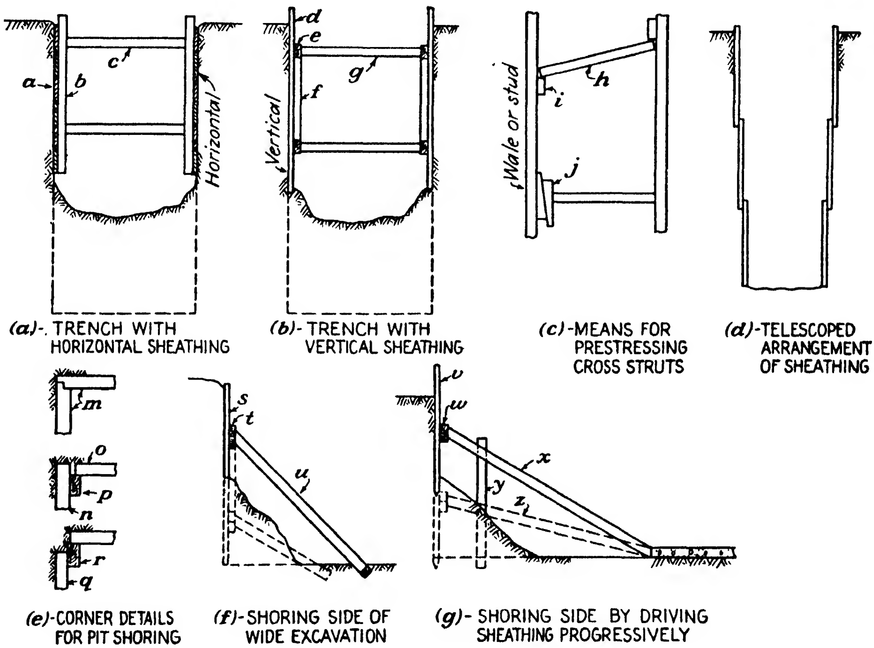


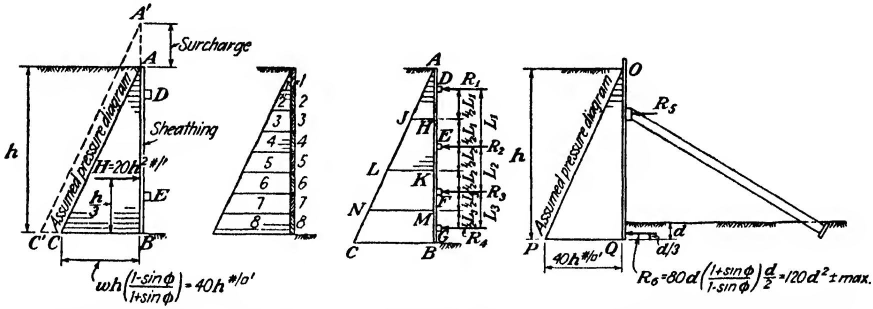
FIG. 11-1. Some arrangements of shoring for sides of excavations.

between *o* and *n* to drive *n* sideways; corner pieces *p* can be driven against *o* to force the latter into bearing; then *p* can be nailed in place. Another method is to use sides *q* of proper length and to wedge both sides at a corner against the soil by wedging and nailing blocks *r*.

(*f*) Many shallow excavations are too wide to justify cross bracing. As shown here, sheathing *s* may be set vertically outside of wales *t* and diagonal braces *u*. As the edge of the excavation is completed, the sheathing can be driven down, and more wales and braces can be added as necessary to make sure that the sheathing cannot rotate or fail in any way.

(*g*) This shows a somewhat similar scheme used for supporting the sides of a deeper large excavation. Steel sheet piling *v* is driven down first to sufficient depth for toe hold below the finished grade. The central part of the excavation is completed and that portion of the concrete mat is poured. Then the sides are excavated, and the wales and bracing members are erected, the latter bearing against the concrete. Sometimes, posts y may be needed to support the long diagonals x and z.

There are many variations of detail, but the danger to be guarded against is the erection of shoring so loose that it permits sufficient movement of the earth to destroy its supporting power under whatever adjacent structure is to be protected. This movement may occur during the dismantling of the



(a)-PRESSURE DIAGRAM PER LINEAR FOOT (b)-PRESSURE ON HORIZONTAL SHEATHING (c)-PRESSURE ON WALES OR STRUTS (d)-PRESSURE ON BRACES AND EMBEDDED TOE PER LINEAR FOOT

FIG. 11-2. Assumed pressures on shoring.

shoring, before its erection, or during its period of service. Therefore, provision for easy and safe conduct of all phases of the operation is essential.

Shoring should be strong and stiff. The minimum probable force for use in its design is the active lateral pressure of the restrained earth. When shoring is pressed against the face of the cut, a portion of the passive earth pressure will be developed, the amount depending upon the severity of the wedging. If the active pressure can be resisted with a safety factor of at least 2, this reserve will usually take care of the excess when good judgment is used in the erection of the shoring.

An attempt to compute with exactness the forces affecting shoring is impractical. Broad safe approximations, easily made, are sufficient except in very special cases. The data shown in Fig. 11-2 are believed to be conservative and sufficient for ordinary use, and the following comments regarding the illustrations are for general guidance:

(a) This gives the pressure diagram, assuming an angle of repose of 30° and a unit weight of earth of 120 p.c.f. This does not include any water pressure because the sheathing is not assumed to be watertight. Surcharge is to be treated as indicated by $A'C'$ in the diagram.

(b) Horizontal sheathing is assumed to resist the force represented by that portion of the pressure diagram which acts directly against it, or that part which is bounded by lines midway between adjacent members if the planks are not in contact.

(*c*) When a series of horizontal wales or braces is used, assume that the pressure upon each is represented by the portion of the pressure diagram between midspan points (or ends) on each side of a given member, neglecting the effect of continuity. Long heavy wales permit the shifting of struts to accommodate excavation operations.

(*d*) When sheathing is braced in part by driving it down a little way to get a toe hold as shown here, assume the pressure diagram to be *OPQ*. The maximum reaction R_6 is limited by the passive resistance of the earth for the depth d in front of the toe. The theoretical resistance R_6 is reduced purposely, as indicated, to allow for light loose saturated soils, assuming a unit weight of 80 p.c.f. for the soil. It is satisfactory to assume that R_6 acts at $\frac{1}{3}d$ from the bottom Q when the embedded depth is too small to offer more than a shearing resistance.

11-3. Single-wall cofferdams in general. The word *cofferdam* denotes a temporary structure used to protect an excavation against lateral earth pressure and water during the process of excavation and of building the foundation or structure that goes in it. In almost all cases, the cofferdam is removed when no longer needed. Sometimes the excavation inside the cofferdam is not unwatered, but generally one of the most important functions of this temporary structure is to facilitate conduct of the work "in the dry." To a certain extent, a cofferdam is similar to shoring except that it is much larger, is made of heavier members, and is generally installed (at least in part) prior to the making of the excavation.

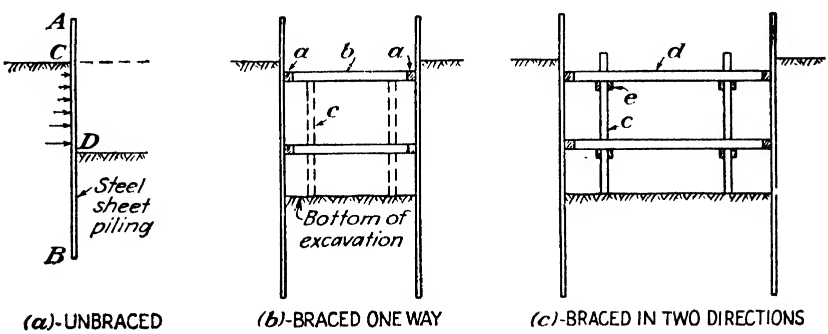


FIG. 11-3. Single-wall cofferdams.

One simple type of cofferdam is shown in Fig. 11-3(*a*). It consists of a single wall of steel sheet piling which encloses an area and which is driven into the ground below the bottom of the excavation far enough to enable the earth to hold the piling in place. The lateral pressure of the ground above D bends the piling as a beam that is cantilevered above this vicinity

The supporting soil is not rigid, hence the upper resultant reaction is below D. The lateral pressure is assumed to be hydrostatic in character. Some suggestions for the assumed unit horizontal pressures caused by various materials acting on cofferdams are given in Table 11-1. However, the most uncertain part of such a cofferdam is the resistance of the soil below D.

To avoid serious uncertainty when deeper excavations are necessary, it is desirable to use one or more planes of bracing to support the sheet piles, as indicated in Fig. 11-3(b). If the cofferdam is small, wales a may be used

TABLE 11-1. Estimated Horizontal Active Pressures Acting upon Cofferdams, Level Top
(Pounds per square foot per foot of depth)

Material	Moist condition	Saturated condition*
Fresh water................	....	62.5
Salt water.................	....	64
Partially fluid mud or muck....	....	75
Ordinary silt................	35	85
Uniform fine sand...........	30	85
Well-graded sand...........	35	85
Well-graded gravel.........	30	85
Soft clay..................	25	80
Dense clay.................	0–25	65

*Includes water pressure and earth pressure. Deduct 60 to estimate earth pressure alone.

alone with the corners mutually supported. Long narrow structures may have a series of struts b to brace the long wales. Larger cofferdams have to be braced both ways, one scheme being the double member one shown in (c). It is necessary to support the bracing vertically by posts c, by using diagonals between the upper and lower members to form trusses that span the opening, by having seat angles tack-welded to the inside of the piling, or by some other device. Lower sets of bracing may be erected as the excavation proceeds but, if so, it is desirable to plan the arrangement so that the members form a honeycomb pattern with cells 8 to 10 ft. or more wide through which the excavation can be conducted. If the depth of water is sufficient, all or part of the wales and their bracing may be fabricated and floated to the site as a unit, then the sheet piling may be driven around it.

The pressures given in Table 11-1 are likely to cause dispute. They are believed to be conservative and are based upon the following reasoning:

1. Assume that a sheet pile wall AB of Fig. 11-4(a) is driven through water into dense clay. Then large rectangular blocks of stone a, b, and c are set outside the wall as shown. The water will still fill the joint between

the piling and the stones and cause full hydrostatic pressure against the wall. The stones do not tend to fall down, hence they cause no added pressure to the wall (neglecting any effect of compression of the clay beneath them).

2. Assume that, as in Sketch (*b*), the big stones are now removed and replaced by a mass of heavy solid spheres. The water will still cause pressure against the piling because the spheres cannot seal it off. Furthermore, the portion of the mass of spheres above some surface of failure *CD* will fall down as soon as the piling is removed. They must therefore cause

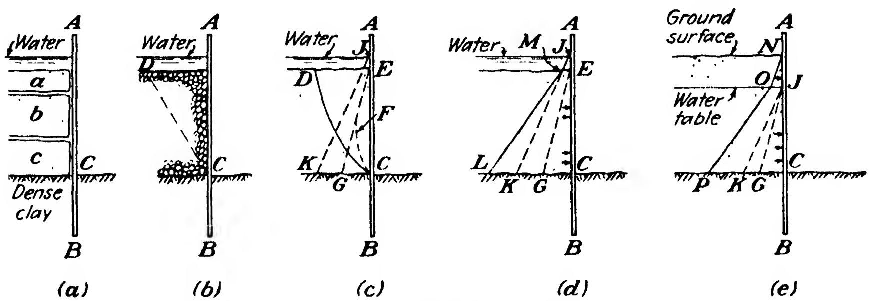


FIG. 11-4. Explanation of assumed pressures on cofferdams.

some additional sideward pressure against the piling if the latter prevents this failure.

3. If the spheres are removed and replaced by a granular soil, as in (*c*), the water will fill the voids and will, for practical purposes, cause the same hydrostatic pressure against the piling. Some wedge of earth like *ECD* will fall as soon as the piling is removed. It, too, must therefore have a supporting reaction provided by the piling if this failure is prevented. The pressure diagram for the earth may be anywhere from something like *EFC* to *EGC*; that of the water, *JKC*. Both act simultaneously.

4. In Sketch (*d*), *JKC* again represents the water-pressure diagram. That for the earth pressure is pictured as *EGC* where *GC* is an assumed hydrostatic pressure given by the Rankine formula and caused by a granular mass having a unit weight equal to that of the submerged soil, and having a coefficient of friction that is applicable for this saturated ground. For example, assume that a fairly uniform fine sand has 40 per cent voids, and $\phi = 30°$. The solid matter when submerged would weigh about $0.6(165 - 62.5) = 62$ p.c.f. Then the intensity of the soil pressure is $p_h = 62\left(\frac{1 - 0.5}{1 + 0.5}\right) = 21$ p.s.f. per ft. of depth. This added to 62.5 equals approximately 84 p.s.f.

5. When the watertable is below the ground line, the pressure diagram *NOPC* in (*e*) is computed similarly, except that *OJ* for moist soil is superimposed upon the pressures from water and saturated ground.

6. Cohesive soils may be able to prevent full contact of the water against the sheet piles. However, this is unreliable because of the violent disturbance of the ground in the immediate vicinity of the piles as they are driven. Stiff clays tend to stand up fairly well temporarily because of cohesion and friction, so that it seems safe to assume that the pressure caused by them on the piling is approximately that caused by the water alone.

One of the serious troubles that may occur in connection with cofferdams is unwatering them. The soil inside may be excavated to the desired depth without removing the water. The interlocks of the piling are then likely to be somewhat loose. As soon as pumping has lowered the water level inside the cofferdam sufficiently, the external pressure tends to pinch the interlocks together so that leakage through the pile walls becomes small. However, as the water level inside is lowered, the unbalanced pressure increases and tends to cause leakage up through the bottom, or to blow the bottom up.

As illustrations of such troubles, refer to Fig. 11-5. In Sketch (*a*), the piling penetrates into deep sand and gravel. The inside is to be excavated and unwatered to *DD*. The gravel is very porous, and an almost infinite amount of water can percolate through the silt, sand, and gravel so that unwatering may be practically a hopeless job. As the water moves through the gravel, the fine particles of sand are likely to be carried with it. Prolonged pumping may then undermine portions of the gravel within the cofferdam and even outside it. The finer the materials are, the greater is this danger of pumping out the soil with the water. In the case of Sketch (*a*), grouting of the gravel inside the piling might be one way of decreasing leakage sufficiently. Lengthening of the piling would avail little.

In Fig. 11-5(*b*) the piling penetrates into deep dense clay. For temporary purposes, the clay is practically impermeable. When the cofferdam is unwatered, the water outside cannot get through the clay, neither can it generally work along the junction between the sheet piles and sticky clay. When the clay is deep, a moderate embedment *DB* of about 20 to 40 per cent of *CD* will generally be safe, although experience with particular soils is the best guide. On the other hand, if a stratum of porous material lay slightly below *B* and if water could penetrate into this stratum from some source, the unbalanced hydrostatic pressure under the plug of clay *BDDB* might heave up the latter—blow up the bottom. There should be enough weight of clay to counterbalance the uplift, or deeper piling should be driven down to cut off the dangerous layer.

Figure 11-5(*c*) pictures another case. Here the structure is to be founded upon the rock. The piling is driven through dense clay to rock, automatically cutting off the sand stratum and sealing the end of the piling. The clay also serves as a seal against the rock. Leaks through seams in the rock are not likely to be serious, but this possibility should be investigated in advance by making several core borings. As long as the clay is sufficiently dense and thick, this situation is favorable to pumping.

A general situation that is more frequently encountered is pictured in Fig. 11-5(d). The structure is to be founded on the rock, which is overlain

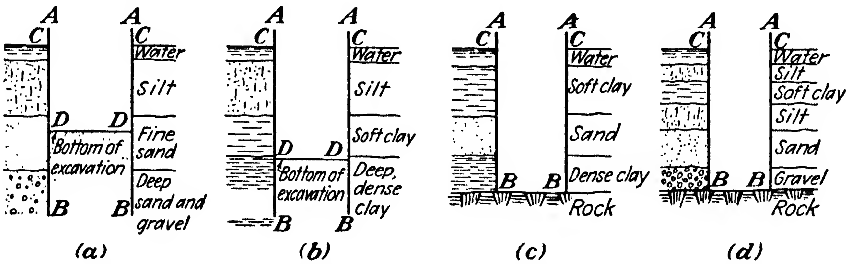


FIG. 11-5. Illustrations of some problems involved in unwatering cofferdams.

by porous water-bearing sand and gravel. If the rock is irregular or if the gravel has a considerable amount of boulders in it, the junction between the piles and the rock will not be watertight. Pumps may or may not be able to handle the leakage. It might be desirable to grout the gravel outside the pile tips or put in a sealing plug of tremie concrete several feet thick, before pumping. The latter should be heavy enough to prevent uplift.

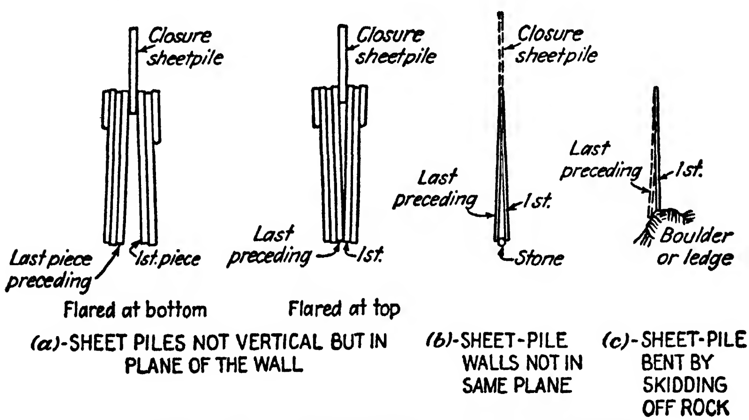


FIG. 11-6. Some sources of trouble in closing sheet-pile walls.

Figure 11-6 shows another source of difficulty with a cofferdam. If the sheet piles are driven clear down, one at a time, there may be trouble in completing the enclosure because of jamming of the last members and interlocks. This can best be prevented by starting all the sections and driving them lightly until the enclosure is complete, then driving each piece in series a few feet, going around the wall as many times as may be necessary.

The great cost, as well as the possible loss of life, that would result from the failure of a cofferdam is too serious for one to take chances with such structures. Not only should they be strong but they should be stiff. The ground outside should not be allowed to slip or start to fail. If it does, the pressures will probably be considerably larger than those assumed here. Wedges and jacks should be used to make sure that all wales and bracing are pressed into proper bearing. Members should be strong enough to resist the extra loads caused by their use to support equipment, by excess pressures created by wedging and jacking, and by bumps from buckets and other equipment.

TABLE 11-2. Estimated Horizontal Passive Resistances of Earth Masses, Level Top
(Pounds per square foot per foot of depth)

Material	Moist condition	Saturated condition*
Ordinary silt	250	210
Uniform fine sand	300	250
Well-graded sand	375	280
Well-graded gravel	425	325
Soft clay	250	170
Dense clay	325	250

*Does not include water pressure. Add 60 to estimate the combined pressure of earth plus water.

11-4. Cantilevered single-wall cofferdams. Assume a section 1 ft. wide of the cantilever piling AB shown in Fig. 11-3(a), with saturated soil outside at C and inside at D. This is reproduced in Fig. 11-7. Let p_a equal the unit active pressure caused by the saturated soil; p_p, the unit passive pressure or resistance; p_n, the difference $p_p - p_a$. Line DF to some scale equals $p_a h$. Below D, the active pressure on the inside annuls any further increase of that outside. Therefore, $CDBGF$ is assumed to represent the net pressure diagram acting on the outside of the piling.

When the soil is saturated, the pressure outside includes that from the water itself and the ground. The values given in Table 11-1 combine them. If the cofferdam is not unwatered, the external pressures should be reduced for the soil alone. This is assumed to be 60 p.s.f. per ft. less than the pressures given for the saturated conditions. Furthermore, the passive pressures given in Table 11-2 for saturated soils are those produced by the soil alone. When the water pressure assists them, add 60 p.s.f. per ft. to the values given in this table.

The earth inside the cofferdam from D to B will resist the tendency of the piling to rotate clockwise in the figure, and it will do so as efficiently as possible. It therefore seems reasonable to assume that, at and somewhat below D, the soil will resist to its maximum—its passive resistance. Since a pressure equal to the active pressure has already been utilized in the determination of line FG, the net resistance p_n is all that can be depended upon. At D, this resistance will start to oppose the active pressures. Line DN, drawn at the slope determined by p_n times the distances, intersects FG at K, a point of zero pressure on the piling. Below K, KN, and KG picture the limit of the resisting intensities of p_n on the right.

Since there is a moment trying to overturn the piling, equilibrium requires that a counteracting moment must exist. It is reasonable to assume that

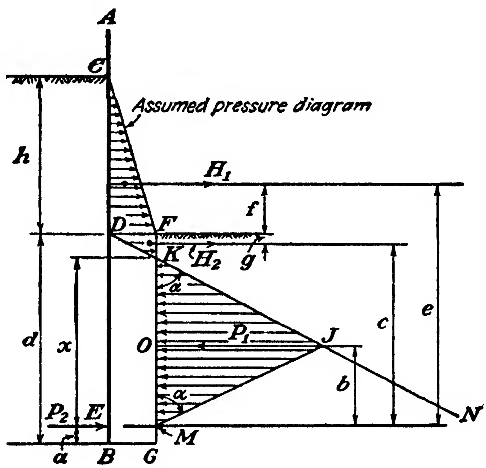


FIG. 11-7. Development of method of analysis of cantilevered cofferdam.

the passive pressures on the right below K and the opposite passive pressures on the left near the tip B of the piling constitute a force couple that causes equilibrium. The forces near K can be the pressures p_n times the corresponding depths, but they will be no larger than necessary. Those on the left near B can be equal to pressure p_n times the height $(h + d)$. This is such a large intensity that the resistance may be assumed to be almost a concentrated force P_2 applied a short distance above B, its magnitude being what is necessary but not exceeding the passive resistance. Below K, the pressures on the right probably increase as fast as they can at first, then increase at smaller rates, and finally decrease again toward the vicinity of B. The pressures on the right probably cannot increase steadily toward B, then suddenly reverse to the left side.

The upper resultant passive resistance below K will be called P_1. It is assumed to be represented by the isosceles triangle KJM of Fig. 11-7, where $MJ = KJ$. The base x is determined by K and the distance a assumed in the location of P_2. If the values of P_1 and P_2 can cause equilibrium without exceeding their respective limiting passive pressures, the piling will not tip over; if their full strengths are not needed, the design is safer, and it should be so. In all cases, $\Sigma H = 0$ and $\Sigma M = 0$, if there is equilibrium.

The procedure for analysis of the piling under the preceding assumptions is the following:

1. Assume distance $a = 1$ ft.
2. Compute the pressures and centers of gravity of portions of the pressure diagrams:

$$H_1 = CDF = p_a \frac{h^2}{2} \qquad H_2 = DFK = \frac{p_a h}{2}\left(\frac{DF}{p_n}\right)$$

$$P_1 = KJM = (d - FK - a)\left(\frac{d - FK - a}{2}\right) p_n \times \frac{1}{2} = \frac{x^2}{4} p_n$$

3. Assume the moment at $E = 0$. Compute lever arms of the forces about this point:

$$b = \frac{x}{2} \qquad c = d - a - \frac{FK}{3} \qquad e = d - a + \frac{h}{3}$$

4. By moments, compute P_1 required:

$$H_1 c + H_2 c - P_1 b = 0$$

$$P_1 = \frac{H_1 e + H_2 c}{b}$$

If this computed value of P_1 is less than $x^2 p_n/4$, not all the resistance of the earth is needed. In fact, the latter should provide a safety factor of perhaps 1.5.

5. Test the required magnitude of P_2:

$$\Sigma H = 0 \qquad H_1 + H_2 + P_2 - P_1 = 0$$

where P_1 is the computed required magnitude. If this computed magnitude of P_2 is less than approximately $p_n(h + d - a)2a$ and is less by the desired safety factor, the structure will be safe in this respect.

6. Compute the shears and bending moments in the piling by using the forces H_1, H_2, P_1, and P_2 as computed. The ordinate OJ of KJM should be reduced to make the area of the triangle equal to P_1. It may be that the real pressure diagram will be curved with a maximum above JO, but this would cause less bending than the assumed diagram. The structure will therefore be safe.

Cantilevered cofferdams can be used to advantage in ordinary earth, but other methods may be applicable and more economical for such conditions. In saturated and submerged ground they are particularly needed. The water adds greatly to the outside pressure and reduces the abutting resistance of the soil inside because of buoyancy, as indicated in Table 11-2. The safe height h of Fig. 11-7 is then greatly restricted. These facts will be illustrated in three examples.

Referring again to Fig. 11-7, it is possible to set up an equation to determine the minimum depth of penetration for any particular soil conditions, having the cofferdam safe under the assumed pressure conditions. The procedure is the following:

1. Determine DF, FK, H_1, H_2, and α for the assumed properties of the soils. Compute distances f and g to the centers of gravity of H_1 and H_2.
2. Assume the safety factor (S.F.) desired against overturning, and multiply the expression for the required P_1 by it.
3. Set up the equation for moments about E in terms of x:

$$H_1(f + FK + x) + H_2(FK - g + x) - \left(\frac{1}{\text{S.F.}}\right)\left(p_n \frac{x}{2}\right)\frac{x}{2}\frac{x}{2} \qquad (11\text{-}1)$$

Solve this by trial.

This last procedure is preferable for design; the preceding one, for analysis.

Example 1. Assume that the sheet piling in Fig. 11-8(*a*) is driven through moist silt and well-graded sand as shown. Assume $p_a = 35$ for both materials; $p_p = 250$ for silt; $p_p = 375$ for sand. Is the piling stable, and what is the maximum bending moment in it?

The computations of pressures and distances are given in Fig. 11-8(*b*). Using these, and taking $\Sigma M = 0$ about E,

$$P_1 b = H_1 e + H_2 c$$

$$P_1 \text{ needed} = \frac{3{,}940 \times 29 + 405 \times 23.5}{11.25} = 11{,}000 \text{ lb.}$$

This is much less than the available P_1 of 43,000 lb.

P_2 needed $= P_1 - H_1 - H_2 = 11{,}000 - 3{,}940 - 400 = 6{,}660$ lb.
P_2 available per sq. ft. of area $= 375 \times 24 + 250 \times 15 = 12{,}800$ p.s.f. (safe)

Assuming a revised value of $\alpha = 340 \times (11{,}000/43{,}000) = 87$ instead of 340 p.s.f. per ft., the point of zero shear and greatest bending would be

$$H_1 + H_2 - 87 \times \frac{y^2}{2} = 0$$

$$y = 10 \text{ ft. below } K$$

Notice that the full value of p_n is used in computing H_2.

$$M_y = 3{,}940 \times 16.5 + 405 \times 11 - \frac{87 \times 10^2}{2} \times \frac{10}{3} = 55{,}000 \text{ ft.-lb.}$$

As stated previously, the diagram for P_1 is likely to have its maximum ordinate above OJ, and this would yield a smaller computed bending moment.

Example 2. Assume the same conditions as pictured in Fig. 11-8(a) except that the watertable is 5 ft. below the top of the silt. Assume that p_a for the moist silt is 35; p_a for saturated silt, 85; p_p for the saturated sand, 280; p_n for the sand, 280 − (85 − 60) = 255. Will the cofferdam be safe?

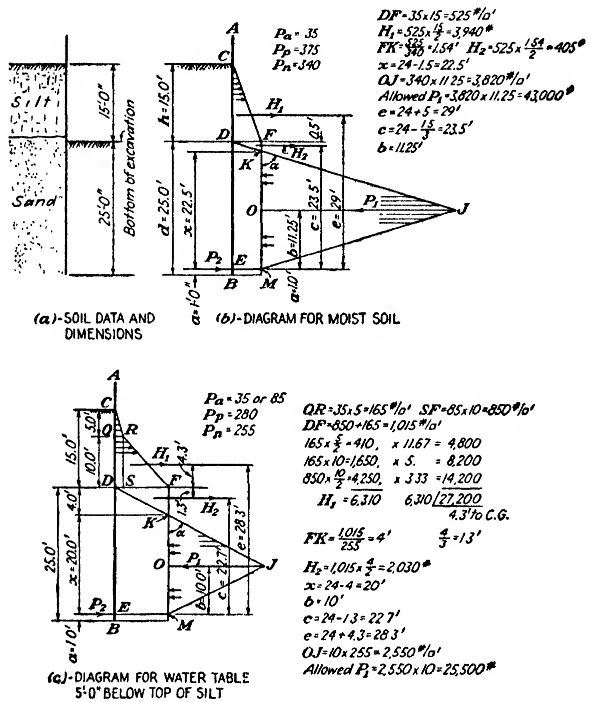


FIG. 11-8. Analysis of a cantilevered cofferdam.

The new pressure diagrams, and the computations for pressures and dimensions, are given in Fig. 11-8(c). Assuming these data,

$$P_1 \text{ needed} = \frac{6{,}310 \times 28.3 + 2{,}030 \times 22.7}{10} = 22{,}500 \text{ lb.}$$

This is slightly less than the 25,500 lb. available. Hence the piling is safe against rotation, but the safety factor is only 25/22 = 1.13.

$$P_2 = 22{,}500 - 6{,}310 - 2{,}030 = 14{,}160 \text{ lb. required}$$

The value of P_2, counting a 2-ft. width, is, approximately,

$$39p_n \times 2 = 39 \times 255 \times 2 = 19{,}900 \text{ lb.}$$

Example 3. Assume the moist soil and conditions given in example 1 and Fig. 11-8(a). Compute the minimum embedment of the piling to give a safety factor of 1.25. Assume the active pressure diagram $CFKD$ in Fig. 11-8(b).

Let KJM represent the pressure diagram for P_1. Use Eq. (11-1)

$$H_1(f + FK + x) + H_2(FK - g + x) - \left(\frac{1}{\text{S.F.}}\right) p_n \frac{x^3}{8} = 0$$

$$3{,}940(5 + 1.5 + x) + 405(1.5 - 0.5 + x) - \left(\frac{1}{1.25}\right) \times 340 \frac{x^3}{8} = 0$$

$$x = 13.6 \text{ ft. (approx)}$$

The embedment then equals, approximately,

$$FK + x + 1.0 = 1.5 + 13.6 + 1.0 = 16.1 \text{ ft. (call it 16)}$$

11-5. Single-braced single-wall cofferdams. When the unbalanced head on a single-wall cofferdam is to be large, the top should be braced. Consider for the present that this bracing is limited to a single layer. This

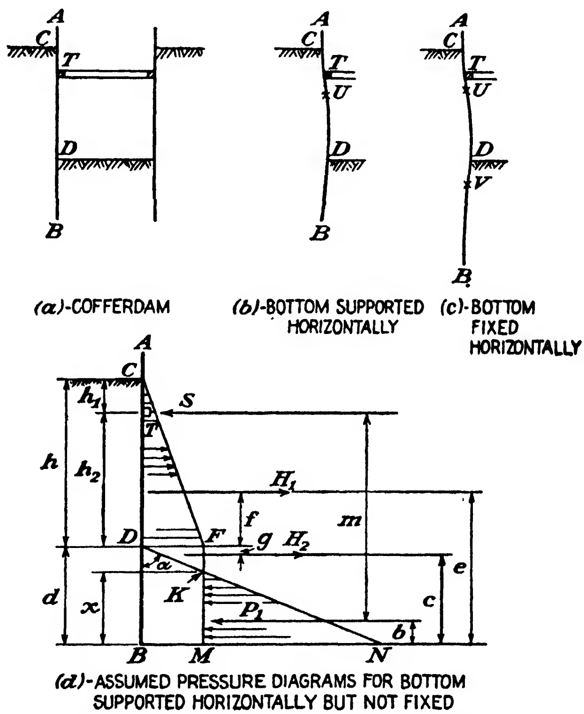


FIG. 11-9. Single-braced, single-wall cofferdam.

produces a structure similar in action to the tied bulkhead of Fig. 10-39 except that the bracing takes the place of tie rods and anchorages. The same method of analysis of pressures on and bending moments in the piling may be used for both types of structure.

Figure 11-9 shows such a cofferdam wall with a wale and brace at T. The end DB is embedded in the ground. If this embedment is small, the piling may bend somewhat as shown in (*b*) with a point of contraflexure at U. As the length of DB is increased, the ground restrains this portion more fully as in (*c*), and it will cause another point of contraflexure at some point V. The pressures and the bending are somewhat different in these two cases.

Assume that the lack of leakage problems permits the use of a short embedment that is just strong enough to prevent the bottom of the piling from kicking inward. In Fig. 11-9(*d*), let $CFMB$ represent the diagram of active pressures as partly counteracted by the earth inside and below D. Let DN be drawn at the angle α determined by p_n since the passive pressure below D will be developed as fast as possible. Assume that KNM represents the diagram of effective passive pressure, with the resultant P_1. Both S and P_1 are passive forces and will be no larger than necessary to produce equilibrium.

To determine the distance x and the force P_1 in (*d*), assume a section of wall 1 ft. long, and take moments about T. Then,

$$H_1(h_2 - f) + H_2(h_2 + g) - \frac{1}{\text{S.F.}}(p_n x)\left(\frac{x}{2}\right)\left(h_2 + FK + \frac{2x}{3}\right) = p_a \frac{h_1^2}{2} \times \frac{h_1}{3} \tag{11-2}$$

As usual, the term S.F. is the safety factor desired. Solve this equation by trial. Then,

$$P_1 = p_n \frac{x^2}{2} \tag{11-3}$$

This gives a minimum value for $d = FK + x$.

In order to estimate S, assume that P_1 represents the position of a reaction point with negligible bending, and treat KNM as the concentrated force P_1. Take moments about this point.

$$H_1(c - b) + H_2(c - b) = Sm = S(h_2 + d - b) \tag{11-4}$$

This is usually a close enough estimate of the pressure per foot on the wale. The load on a strut (or tie) is then S times the spacing of the supports.

Let Fig. 11-10 represent the case in which avoidance of leakage requires deep embedment of the piling. As in the case of cantilevered piling, assume that $CFME$ is the diagram of active pressures; KJM, of passive resistances. The point of inflection V in Fig. 11-9(*c*) may be somewhere near the point of zero pressure K in Fig. 11-10. As a first trial, assume them to coincide. Taking moments about K,

$$H_1(FK + f) + H_2(FK - g) = Sn = S(FK + h_2) \tag{11-5}$$

Assuming this value of S, take moments about E, the assumed line of action of the passive resistance P_2.

$$H_1e + H_2c - P_1b - S(h + d - h_1 - a) = 0 \tag{11-6}$$

Compare the required P_1 with the maximum allowable $p_n(x^2/4)$. If P_1 is safe, draw a revised diagram KJM to represent its required magnitude, letting DK remain unchanged. Now compute the moment diagram for the bending in the piling under the action of these forces. If the required P_1 is larger than that allowed by the assumed properties of the soil, the ground will probably yield slightly near D, and the point of inflection may move

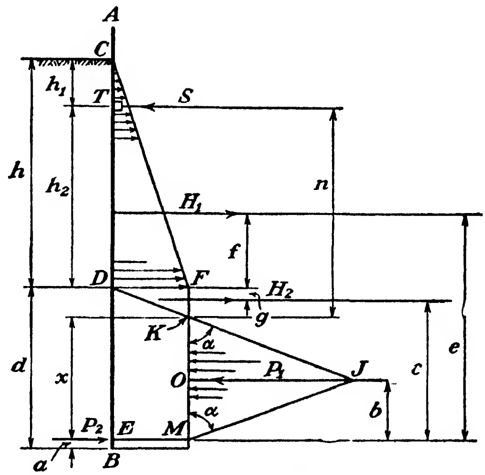


FIG. 11-10. Assumed pressure diagrams for cofferdam with bottom fixed horizontally.

downward. However, it is probably desirable to place the struts in a lower position or to add more bracing. In other words, if it is desired to have the earth restrain the piling fully, at least one should design the structure so that, according to the computations, it can do so.

When the proportions of the structure make one suspicious that the point of inflection will not be close to the assumed point K, repeat the calculations with the assumption that it is 2 or 3 ft. higher, then 2 or 3 ft. lower. Proportion the structure and its bracing so that it is safe under any of these conditions. Check to see that the conditions $\Sigma H = 0$ and $\Sigma M = 0$ for equilibrium are reasonably satisfied. Exact agreement is not essential. When part of the diagram KJM enters the equation for S, use it as the approximated one found after the solution of Eq. (11-6).

The results of these computations will not be exactly correct, but they seem to give results as accurate as the unavoidable uncertainties in the assumed data warrant.

A cofferdam may have to be used in sloping ground. The pressure on the high side will then exceed that on the low one. This unbalance should be provided for. Ordinarily this can be done easily because the bracing will transmit the larger force to the downhill side where the sheet piles will develop the passive resistance of the soil as far as necessary. This, however, may change the maximum bending moments in the piling.

Example 1. A cofferdam is to be built as shown in Fig. 11-11(a). Assume p_a = 80 p.s.f. per ft.; sea water = 64 p.c.f.; p_p = 250; and p_n = 250 − (80 − 60) = 230. Find the absolute minimum penetration d to hold the bottom of the piling (S.F. = 1), compute the maximum bending moment in the wall, and find the pressure in the struts if they are 10 ft. c.c.

The assumed pressure diagrams are shown in Fig. 11-11(b), with the preliminary computations and computed dimensions. Taking moments about T,

$$H_1(14 - 6.5) + H_2(14 + 2.2)$$

$$- \quad 230x\left(\frac{x}{2}\right)\left(14 + 6.6 + \frac{2}{3}x\right) = 64 \times \frac{6^2}{2} \times 2 \text{ (approx)}$$

$$x^3 + 30.8x^2 = 2{,}440$$

$$x = 8 \text{ ft. (approx)}$$

and

$$d = 6.6 + 8 = 14.6$$

or call x = 8.4 ft. and d = 15 ft.
Taking moments about P_1,

$$H_1(5.6 + 6.6 + 6.5) + H_2(5.6 + 4.4) = S(5.6 + 6.6 + 14)$$

$$S = 12{,}300 \text{ lb.}$$

Shear at S = 12,300 − 1,160 = 11,140 lb. and TZ = 400 p.s.f.

$$\tfrac{1}{2}[400 + (400 + 80y)]y = 11{,}140$$

$$y = 12.4 \text{ ft. for point of } M_{max} \text{ below } T$$

$$M = 12{,}300 \times 12.4 - 800 \times 15.07 - 320 \times 13.4 \times 6.7 - 80 \times 13.4 \times 6.7 \times 4.47$$

$$M = 80{,}000 \text{ ft.-lb. per ft. of cofferdam (very high)}$$

Force in a strut = $10S$ = 123,000 lb.

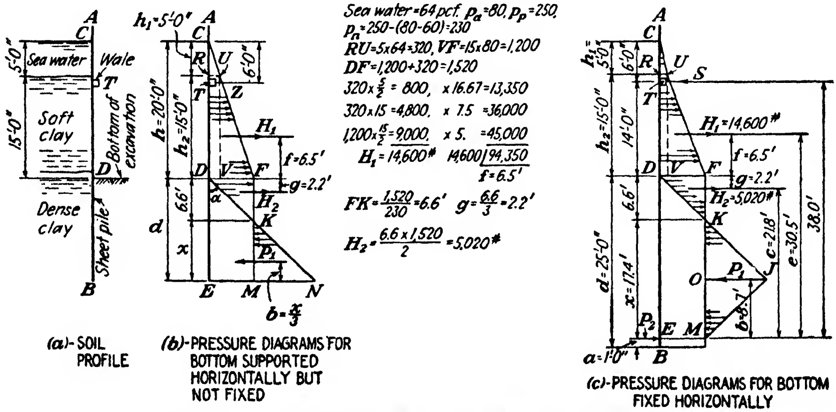


FIG. 11-11. Analysis of single-wall, single-braced cofferdams.

Example 2. Assume the same cofferdam as in Fig. 11-11(*a*) except that fine sand replaces the dense clay, and the piling is to be driven so that $d = 25$ ft. in order to have the bottoms of the sheet piles reach a dense sandy clay. What is the estimated load per foot on the wale at T, and what is the maximum bending moment in the piling? Assume that $p_p = 250$ p.s.f. per ft.; $p_n = 250 - (80 - 60) = 230$ p.s.f. per ft.

Having the same values as for example 1, the diagrams above K in Fig. 11-11(*b*) have been copied in (*c*).

$$x = 25 - 1 - 6.6 = 17.4 \text{ ft.}$$

$$OJ = \frac{17.4}{2} \times 230 = 2{,}000 \text{ p.s.f.}$$

$$P_{1\,\max} \text{ allowed} = 2{,}000 \times \frac{17.4}{2} = 17{,}400 \text{ lb.}$$

Taking the first trial moments about K as a point of contraflexure,

$$14{,}600(6.5 + 6.6) + 5{,}020 \times 4.4 = S(14 + 6.6)$$
$$S = 10{,}300 \text{ lb.}$$

Taking moments about E, in Fig. 11-11(*c*),

$$14{,}600 \times 30.5 + 5{,}020 \times 21.8 - P_1 \times 8.7 - 10{,}300 \times 38 = 0$$
$$P_1 = 18{,}700 \text{ lb.}$$

This is a little over the maximum P_1 allowed under the assumptions.

The point of zero shear is found to be 11 ft. below T. Then,

$$M = 10{,}300 \times 11 - 800 \times 13.67 - 320 \times \frac{12^2}{2} - \frac{80 \times 12^3}{6}$$
$$M = 56{,}000 \text{ ft.-lb. per ft. of cofferdam}$$

This would require heavy Z-shaped piling. More bracing should be added, or the position of the bracing should be lowered considerably.

Test these results in comparison to those found by assuming the point of inflection to be at points other than K, using the same general method of analysis. Provide for what seems to be the worst reasonable case.

11-6. Multibraced single-wall cofferdams. It is generally desirable, or even necessary, to support the sheet-pile wall of a deep cofferdam by several sets of bracing to prevent excessive bending in the piling, and to avoid dependence upon the somewhat uncertain restraint of earth around the embedded ends. The braces may be spaced in whatever way the designer prefers, or as construction procedures in the field require.

In some ways, it is desirable to arrange the bracing so that the maximum bending moments in the piling at various points utilize the strength of the piling and are nearly equal. Bracing is very expensive, and economy is essential. Therefore, piling that has great strength in beam action is desirable for this feature, whereas sections with straight or shallow arched webs may be better for driving, especially for penetrating gravel or soft rock. On the other hand, there are advantages in arranging the wales and bracing so that each layer is composed of corresponding members of the same size instead of having them vary. Deep cofferdams may require heavy steel beams as wales, with steel or wooden struts. For example, one given size of beam may be available and its use throughout may be desired.

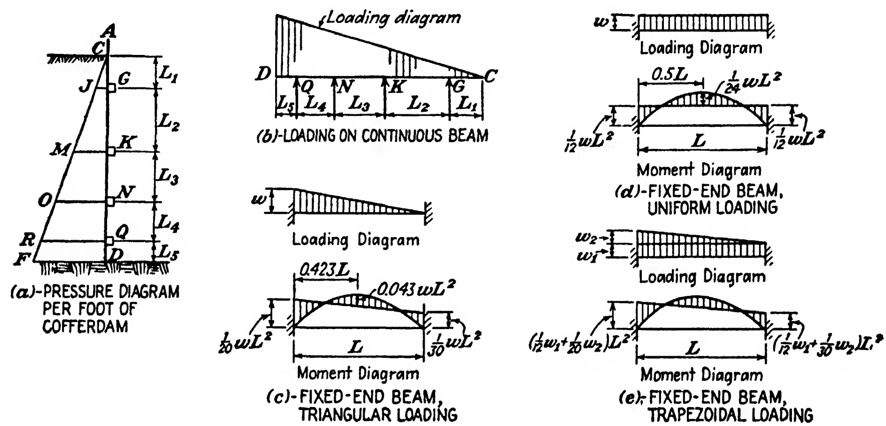


FIG. 11-12. Data for analysis of multibraced cofferdam, cantilevered at top and bottom.

No matter what is planned regarding the equality of bending moments in piling or of reactions on bracing, there are likely to be considerable variations and uncertainties besides those caused by the uncertainties in the combined soil and water pressures themselves. Wedging or jacking of struts and wales may cause some reactions to be larger than the theoretical; some may be smaller. The piling may be pressed against the soil in places so that it develops more than the active earth and water pressure, thus causing higher stresses. In other cases, it may be slack, even to the extent of permitting small movements with unknown consequent pressures because the resisting friction and cohesion have been overcome by such movements. It is also very difficult to drive piling and to erect bracing with absolute perfection. Therefore, conservatism in design is desirable. How, then, should such a cofferdam be designed?

As in the preceding cases, assume that the earth pressure is hydrostatic in character. Let Fig. 11-12(a) represent a cofferdam with four sets of bracing at unequal spacing. The sheet piling is driven down to rock. Since the piling is continuous, it will act like a beam that is continuous over a series of supports, with the top cantilevered above G and the bottom assumed to be cantilevered below Q. For simplicity, represent the piling and its loads as in Sketch (b). The bending moments may now be found by moment distribution, and this method is recommended. For this, the fixed-end moments can be found by means of the data in (c) for triangular loading, and by (e) for trapezoidal loading.

If the use of moment distribution is not desired, a rough estimate of the bending moments may be made as follows, provided the lengths of cantilevers and intermediate spans are proportioned reasonably and successive spans do not change too rapidly:

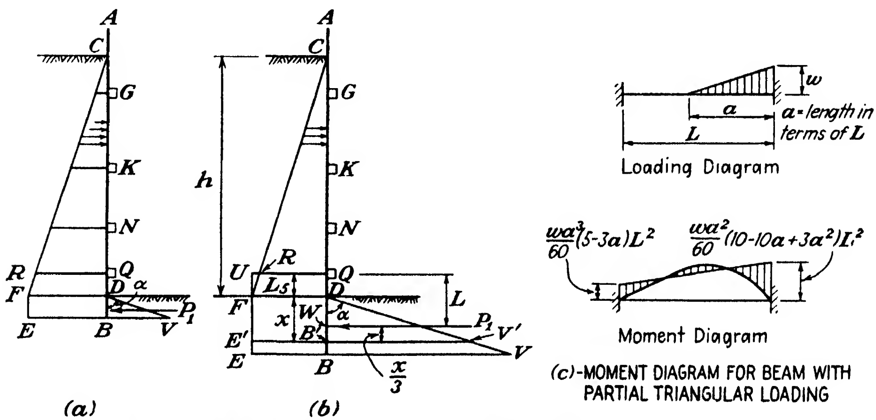


FIG. 11-13. Assumed pressure diagrams at bottom of embedded piling of multibraced cofferdam.

1. Compute the bending moments in the ends GC and QD as cantilevered beams.

2. Assume the piling to be fixed at all bracing supports.

3. Compute the moments at the right and the left sides of each intermediate support K and N.

4. Average the computed fixed-end moments at K and N. Use the cantilever moments at G and Q. If the cantilever moment at a support is small as at G, add 10 to 20 per cent to the average bending moment computed for the next support—K in this case. If the cantilever moment is anywhere near equal to the fixed-end moment at the other side of the support, the moment at the adjacent support need not be increased. Call these the maximum bending moments for design. Those in the portions of the piling between supports will be smaller and need not be computed, unless the lengths of spans are very irregular.

If the end D is supported by the rock and if this support can be trusted, the span QD may be treated as a beam fixed at Q and simply supported at D.

It is sufficient to estimate the reactions or forces upon the bracing as though all intermediate spans of piling were simply supported, with trapezoidal loading. For the end braces, add the full weight of the load on the respective cantilevers.

There are many cases in which a multibraced cofferdam will have the bottom of the sheet piling embedded in soil below D, Fig. 11-13(a). If this embedment is slight and if it cannot completely support the end B, the portion QB may be considered to be a cantilever with the loading diagram

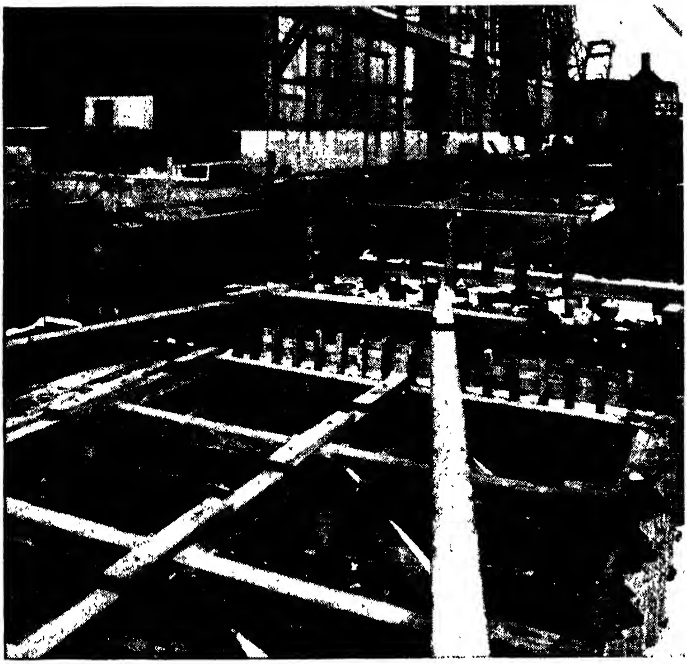


FIG. 11-14A. Cofferdam arranged in cells for excavation to different levels. Use of wooden bracing is shown. (*Courtesy of Westcott & Mapes, Inc., Consulting Engineers, New Haven, Conn.*)

$QRFEB$ for active pressure and DVB for passive pressure. This is seldom the case, however.

When the embedment of DB is considerable, it can support the shear at the bottom of the piling, and perhaps fix that end against rotation. In this case, the bending moment at Q and in the portion QB may be estimated in the following manner, referring to Fig. 11-13(b):

1. Assume the active-pressure diagram to be the rectangle $QUEB$.

2. Assume the passive-pressure diagram to be the triangle DVB, where angle α represents p_n p.s.f. per ft.

3. Assume that the passive pressure will be developed as rapidly as possible to support the piling. Excessive length of DB over that required will be assumed to fix the piling in position but not to add materially to the reaction or the bending moments.

4. Find the distance x such that the minimum passive resistance $DV'B'$ will support one-half of the pressure caused by the diagram $QUE'B'$. Thus

$$\frac{QUE'B'}{2} = DV'B'$$

$$E'B' = p_a h \qquad \text{and} \qquad B'V' = p_n x$$

$$\frac{1}{2} p_a h(L_5 + x) = p_n \frac{x^2}{2}$$

$$p_n x^2 - p_a hx - p_a hL_5 = 0 \tag{11-7}$$

Solve for x.

5. The resultant passive pressure P_1 will act at $x/3$ above B'. Assume this line of action to be the support point for the piling, thus determining the span $L = QW$.

6. Assume the span QW to be fixed at both ends, and compute the fixed-end moment at Q as $\frac{1}{12} p_a hL^2 - (p_n DW/60)a^3(5 - 3a)L^2$, where the latter term is explained by reference to Fig. 11-13(c).

7. Assume the reaction at Q from span QW to be $\frac{1}{2} p_a hL - p_n(DW)^3/6L$.

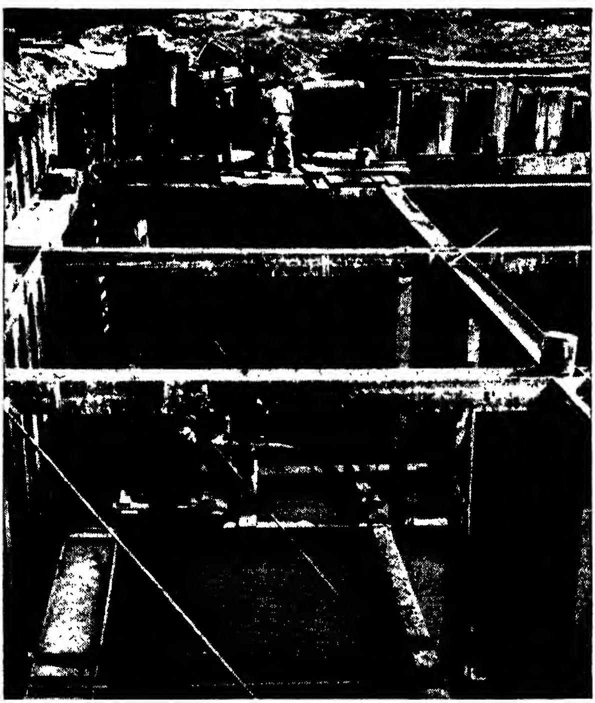


FIG. 11-14B. Cofferdam for Pier 1S, Chesapeake City Bridge, Chesapeake City, Md. The bracing is made of welded steel that can be cut and welded to fit closely in the field. Unwatering cofferdam to surface of tremie-concrete seal on top of piles. Water surface shown at El.-20. Top of seal at El. -31.5 ***(Designed by Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y. Built by Fehlhaber Pile Co.)***

The cofferdam in Fig. 11-15 shows a different solution for supporting the bottom of the piling when excavation is to be carried to rock. Briefly, the procedure is as follows:

1. Drive sheet piles lightly to rock, assemble the bracing as a cage, then sink it to the river bottom.
2. Excavate in the wet to rock.
3. By means of divers, excavate a shallow trench in the soft rock around the inside perimeter.
4. Redrive the sheeting firmly to rock, and push the bracing cage to the desired position.
5. Erect rails as guides for the sliding forms.

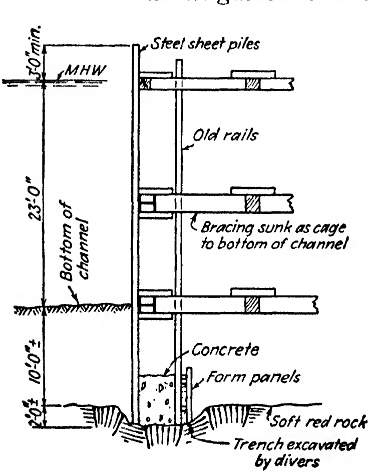


FIG. 11-15. Cofferdam with concrete seal used for Ferry St. bridge piers over Quinnipiac River by C. W. Blakeslee & Sons, New Haven, Conn.

6. Have divers erect forms along the inside of the trench, filling in closing pieces between the main panels.
7. Clean out the rock trench, and place concrete seal 5 by 5 ft. by underwater buckets.
8. Pump out water, and build a sandbag drainage trench to the pumps on top of the concrete wall. This is to collect any seepage through sheeting.
9. Clean off loose rock, then build the pier.

The cofferdam pictured in Fig. 11-16 also deserves examination. It was built to a depth of 60 ft. in saturated volcanic sand. A few points to notice particularly are the following:

1. The bottom set of wales and bracing is separate from the main bracing cage. This was weighted by old rails to overcome buoyancy. It was made with 2-in. clearance and was suspended on wire ropes attached to hand-operated winches so that it could be lowered as fast as excavation proceeded, thus guarding against any sudden failure of the bottoms of the sheeting.
2. Batter piles on the outside held the sheeting in line until considerable excavation in the wet was completed. The main bracing cage was then erected inside at a higher position than the final. Although weighted, it was driven to proper position by jacks that reacted against beams placed across the cofferdam and hook-bolted to the sheeting. The lowest set of bracing also had to be forced down to position because deflection of the piles took up the clearance. This was then strutted against the main cage.
3. Divers placed wedges between the sheeting and wales to ensure bearing.

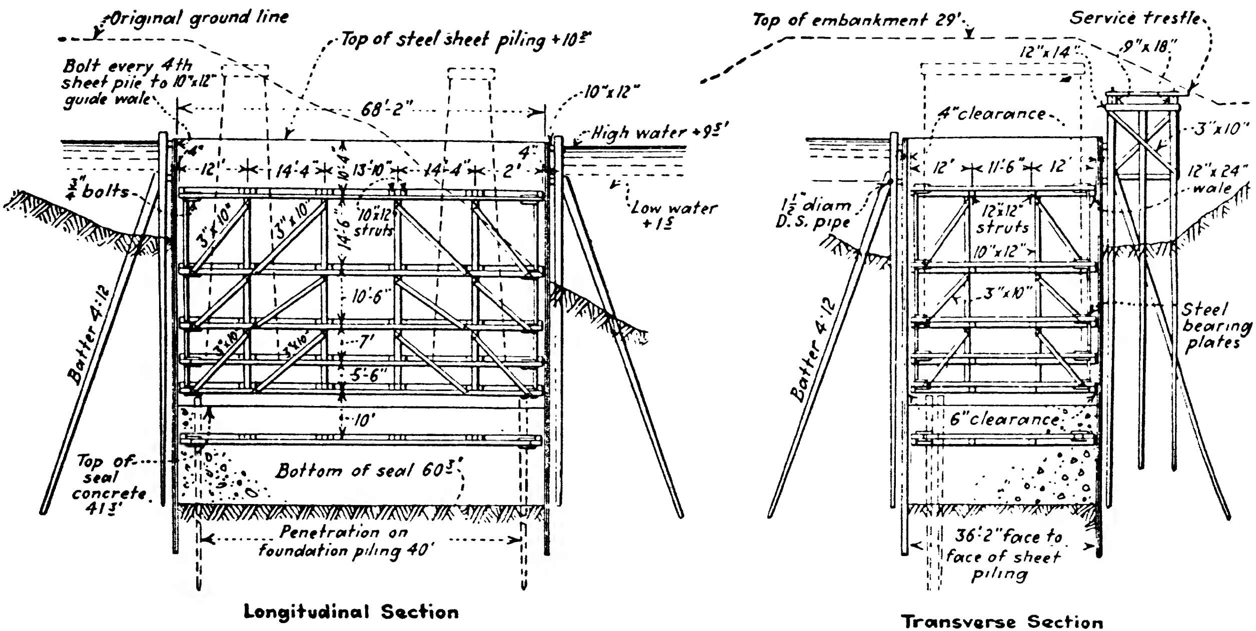


FIG. 11-16. Cofferdam for 60-ft. head for a bascule pier of the Cowlitz River Bridge. Note the interior bracing cage that was lowered as excavation progressed and the bottom "drop set" that was weighted by rails, suspended by six wire ropes over winches, and kept as far down as possible. The guide wales were strongly supported on the outside by piles. *(Courtesy of N.F. Helmers, Cofferdam for 60-ft. head with One Line of Sheeting, Engineering News-Record, July 4, 1935.)*

4. The foundation piles were then driven by a hammer equipped for underwater work, assisted by jetting.

5. The tremie-concrete seal of over 1,800 cu. yd. was then placed in one operation.

6. Unwatering and pier construction then followed.

Example 1. Assume that a cofferdam is to be built as shown in Fig. 11-17(*a*). Estimate the bending moments in the piling at the wales, and the pressure per foot on the wales. Assume $p_a = 85$ p.s.f. per ft. Neglect any lateral support at *D*.

The pressure diagram will be as shown in Fig. 11-17(*b*). The fixed-end moments, as computed from the data in Figs. 11-12(*c*) and 11-17(*b*), are as follows:

Point		*M, ft.-lb.*
G top	$425 \times 2.5 \times 1.67$	= 1,770
G bot.	$(\frac{1}{12} \times 425 + \frac{1}{30} \times 850)10^2$	= 6,370
K top	$(\frac{1}{12} \times 425 + \frac{1}{20} \times 850)10^2$	= 7,790
K bot.	$(\frac{1}{12} \times 1{,}275 + \frac{1}{30} \times 680)8^2$	= 8,250
N top	$(\frac{1}{12} \times 1{,}275 + \frac{1}{20} \times 680)8^2$	= 8,950
N bot.	$(\frac{1}{12} \times 1{,}955 + \frac{1}{30} \times 510)6^2$	= 6,500
Q top	$(\frac{1}{12} \times 1{,}955 + \frac{1}{20} \times 510)6^2$	= 6,800
Q bot.	$2{,}465 \times 3 \times 1.5 + 255 \times 1.5 \times 2$	= 11,900

The stiffness factors—the relative values of I/L of the beams—are given as *K* in Fig. 11-17(*c*). The carry-over factors are shown in the circles. With the fixed-end moments previously estimated, the moment distribution is carried out as shown in (*c*). The summations give the moments for design purposes.

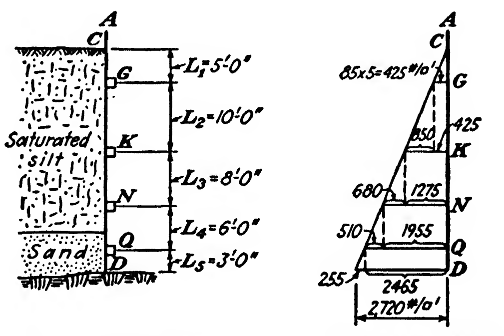
The reactions on the wales are estimated as follows, assuming simple-beam action:

Point		*One end, lb.*	*Total reaction, lb.*
G top	425×2.5	= 1,060	
G bot.	$425 \times 5 + 850 \times 5 \times \frac{3.33}{10}$	= 3,540	4,600
K top	$425 \times 5 + 850 \times 5 \times \frac{6.67}{10}$	= 4,960	
K bot.	$1{,}275 \times 4 + 680 \times 4 \times \frac{2.67}{8}$	= 6,010	10,970
N top	$1{,}275 \times 4 + 680 \times 4 \times \frac{5.33}{8}$	= 6,920	
N bot.	$1{,}955 \times 3 + 510 \times 3 \times \frac{2}{6}$	= 6,380	13,300
Q top	$1{,}955 \times 3 + 510 \times 3 \times \frac{4}{6}$	= 6,880	
Q bot.	$2{,}465 \times 3 + 255 \times 1.5$	= 7,780	14,660

As a matter of curiosity, see how the bending moments computed in Fig. 11-17(*c*) compare with the approximation of averaging the fixed-end moments instead of using the moment-distribution method. The results are the following:

Point	Fixed-end M, ft.-lb.	Averaged M, ft.-lb.	Adjusted M, ft.-lb.
G as cantilever...	1,770	1,770	1,770
K top...........	7,790		
K bot...........	8,250	8,020(+20%)	9,620
N top...........	8,950		
N bot...........	6,500	7,720	7,720
Q as cantilever...	11,900	11,900	11,900

Example 2. Assume that the cofferdam of Fig. 11-17(*a*) now has the sheet piling driven down into stiff saturated clay instead of onto rock. Assume the pressure diagrams to be as pictured in Fig. 11-13(*b*). Then $p_n = 250 - (85 - 60) = 225$ p.s.f. per ft. Find the depth to which the piling should be driven to provide adequate lateral support. Also estimate



(*a*)-SOIL PROFILE AND DIMENSIONS

(*b*)-PRESSURE DIAGRAM PER FOOT OF WALL

	Q K=1.67	(.5)	N K=1.25	(.5)	K K=1.0	(.5)	G	
F.E.M.	-11,900	+6,800	-6,500	+8,950	-8,250	+7,790	-6,370	+1,770
1st Dist.		+5,100	-1,400	-1,050	+260	+200	+4,600	
1st C.O.		-700	+2,500	+130	-500	+2,300	+100	
2nd Dist.		+700	-1,510	-1,120	-1,000	-800	-100	
2nd C.O.		-750	+350	-500	-560	-50	-400	
3rd Dist.		+750	+90	+60	+340	+270	+400	
3rd C.O.		+40	+370	+170	+30	+200	+130	
4th Dist.		-40	-310	-230	-130	-100	-130	
Σ		+11,900	-6,410	+6,410	-9,810	+9,810	-1,770	
M	11,900		6,410		9,810		1,770	

(*c*)-MOMENT DISTRIBUTION

FIG. 11-17. Analysis of a multibraced cofferdam.

the bending moment in the portion QB of the piling, and find the approximate revised reaction on the wale at Q.

From Eq. (11-7), since $p_a h = 2{,}720$ p.s.f. and $L_5 = 3$ ft., find x.

$$225x^2 - 2{,}720x - 2{,}720 \times 3 = 0$$
$$x = 14.5 \text{ ft. (call it 15)}$$

Then $QB' = 15 + 3 = 18$ ft., $QW = 3 + \frac{2}{3} \times 15 = 13$ ft.

$$M_Q = \frac{1}{12} \times 2{,}720 \times 13^2 - \frac{2{,}250}{60} \times 0.77^3(5 - 3 \times 0.77)13^2 = 30{,}600 \text{ ft.-lb.}$$

End shear:

$$\frac{1}{2} \times 2{,}720 \times 13 - \frac{225 \times 10^3}{6 \times 13} = 14{,}800 \text{ lb.}$$

Reaction at Q:

$$14{,}800 + 6{,}880 \text{ (from example 1)} = 21{,}700 \text{ lb.}$$

These computations show that the forces and moments in the bottom wale and the lower portion of the piling are very severe compared to those higher up. Therefore, the estimate of moments and reactions, as computed in example 1, should not be used but should be revised because of the large moment at Q. The trouble comes from the inability of the clay to develop sufficient passive resistance fast enough to make a short effective span below Q.

11-7. Double-wall cofferdams. It is often necessary or desirable to use cofferdams with double sheet-pile walls. One such is shown in Fig. 11-18(a). The need for this generally is not the depth required inside the cofferdam but the greater probability of securing watertightness. A single wall of piling driven to irregular rock, into gravel, or onto boulders will almost inevitably have irregular openings below the ends of the piles through which water can pass in great volume provided a granular water-bearing stratum exists at that position. Grouting outside of the pile tips may be helpful but is not completely dependable.

A double-wall cofferdam is usually built by driving down two walls of sheet piles around the area to be excavated. These walls may be 4 to 6 ft. or more apart in order to have room enough to excavate between them. The earth inside is generally removed by excavation "in the wet," with struts to separate the two rows at top and lower down if necessary. When the bottom of the space is cleaned sufficiently and when the piling is seated firmly in case the removal of boulders makes this necessary, the space between the rows like AB and $A'B'$ of Fig. 11-18(a) is filled with clay. Better yet, the bottom is filled with a layer of tremie concrete several feet thick, the space above it is later unwatered, and then stiff clay is packed between the walls. This provides a dense fairly impervious wall and a reasonably good seal at the bottom.

The next procedure is that of excavating, bracing, and unwatering the cofferdam as usual. If local leaks are discovered under the bottom, grouting outside of the bottom of the structure may be beneficial in sealing them.

Double-wall cofferdams may be used to depths of 70 to 75 ft., whereas single walls should generally be limited to 40 or 50 ft. (or perhaps 60 ft.)

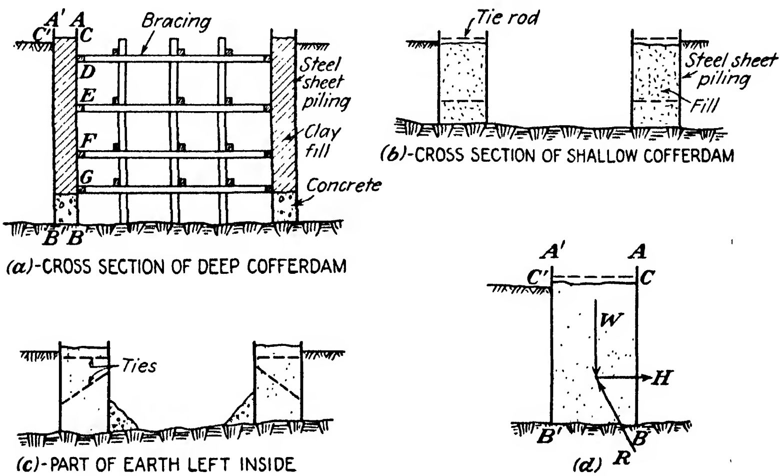


FIG. 11-18. Double-wall cofferdams.

when the excavation goes to rock. With a single wall there is greater danger of a blow-in if the piling hits steeply sloping rock or boulders so that it cannot get a toe hold on the bottom. For deep double-wall cofferdams, the concrete seal on the bottom is very desirable because the mortar is likely to penetrate between any remaining gravel or boulders and fill crevices. The clay, especially when placed through water, is not likely to do this.

The two rows of sheet piles shown in Fig. 11-18(a) should not be assumed to share equally in resisting the outside pressure unless concrete fill or firm bracing is used between them. When merely packed with clay, the row $A'B'$ tends to press against the fill, and the latter in turn transmits the load to AB. In any case, the bracing must resist the total pressure. Both piling and bracing may be designed or analyzed by the methods previously explained for single walls.

Here is a case that emphasizes the safety element in design. A large cofferdam was built to surround the excavation for a large bridge pier to be founded upon sloping irregular rock. The bottom of the piling on the deep side was approximately 70 ft. below M.H.W.; on the landward side, 40 ft. In the interest of economy, the riverward side and most of the two sides normal thereto were made double-wall construction with a concrete seal 10 or 15 ft. deep and clay fill above it, but the rear corners and landward side were made with a single wall. After excavation and unwatering, the bottom portions of some of the single-wall piling near one rear corner failed. This caused a blow-in that caused the loss of lives as well as of money. The intended economy proved to be extremely unwise and expensive.

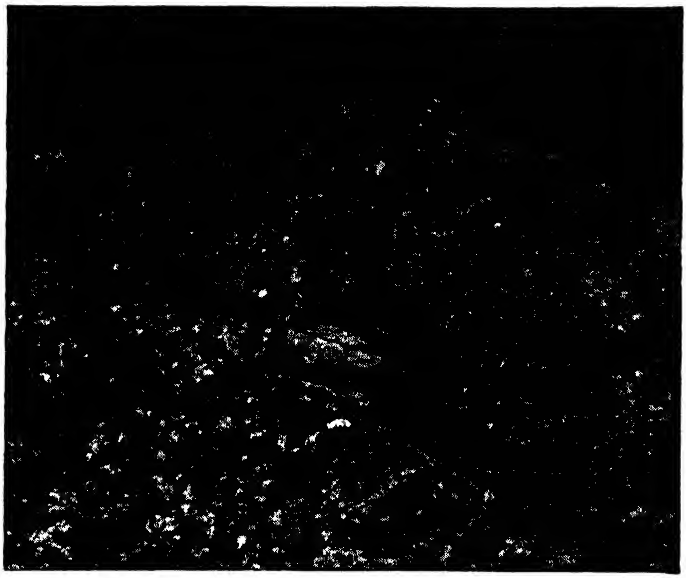


FIG. 11-19. Excavating inside the cofferdam for pier 2S of the Chesapeake City Bridge. *(Designed by Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y. Built by Fehlhaber Pile Co.)*

Another type of double-wall cofferdam for use with shallow excavations to rock is shown in Fig. 11-18(*b*). This was used for the abutments of the Bayonne Bridge. The double sheet-pile enclosure is driven first. The space between the walls may often be left filled with the original earth, the tops of the piles being connected by tie rods between the two walls to prevent spreading. It may then be desirable to leave an embankment or berm of earth along the inside to support the bottom of the inner wall, as shown in Sketch (*c*). This, however, increases the required size of the cofferdam in plan. Another method is to drive the piling, excavate between the walls, brace them temporarily or permanently, tie them together by tie rods and welded connections, and then backfill the space. A mixture of clay and gravel at the bottom will help to prevent leakage along the rock. Generally, when this type is used, the hydrostatic head is small so that the main portion of the backfill may be gravel and sand, or even riprap, that has dependable internal friction.

The stability of the cofferdam of Fig. 11-18 is obtained through its action as a sort of earth dam, with steel sides to provide vertical faces. The forces are pictured in (*d*). Piles *AB* and *A'B'* should be relatively far apart in order to provide adequate weight of fill, shear resistance in the fill, and frictional resistance along the bottom. The cofferdam should be stable as an earthen structure. In general, the maximum height of such a cofferdam should not exceed its width. To test it, assume that it retains its rectangular shape, then check the following:

1. Stability against overturning. Friction along *B'C'* will aid this but is not usually relied upon.
2. Maximum pressure near the toe *B*.
3. Shear vertically in the soil. If the resistance from internal friction and cohesion is too weak, the mass will distort into an acute-angled parallelogram, and it may then fail. A few diagonal tie rods sloping upward toward *C*, or a few struts sloping upward toward *C'*, Sketch (*d*), will be helpful.

The general methods of calculation are illustrated in the next article. However, one of the troubles to be guarded against is the kicking out of the toe of the inside row of piling unless tie rods or a strong berm is used.

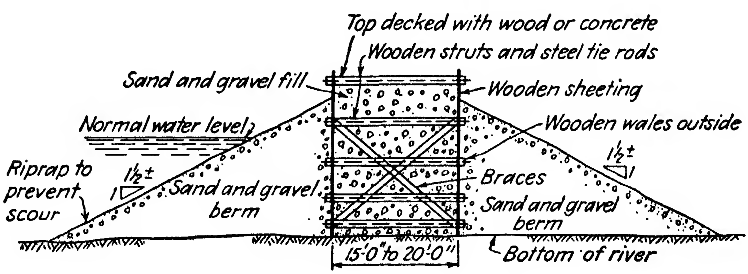


FIG. 11-20. Box-type cofferdam of wood formerly used by U.S. Army Engineers for low heads.

Figure 11-20 shows a wooden cofferdam that was formerly used considerably for work along the Ohio River. It is still a useful type but has been replaced for large heads by those having steel sheet piling. It is obvious that the large inside berm requires a cofferdam very extensive in area.

The cofferdams shown in Figs. 11-21 and 11-22[1] were made with double steel-plate walls, structural steel diaphragms, bulkheads, and deck so that they consist of several tanks that can be unwatered for floating or flooded for sinking. The type in Fig. 11-21 was floated to position and sunk to the bottom where it was landed upon 16 wooden piles that had been driven and

[1] S. M. Koefoed, Four Cofferdams for Forty-one Foundations, *Engineering News-Record*, July 6, 1939.

cut off previously. A sheet-pile ring was next driven around the inside, then the excavation was made under water. Divers placed a series of partitions in the excavated space so that the tremie concrete could be placed in one of the 10 compartments at a time. This gave better results than could be obtained by trying to do the concreting of the entire area at one time. The space between the sheet piles and cofferdam was then sealed with concrete, and the cofferdam was unwatered. After the lower

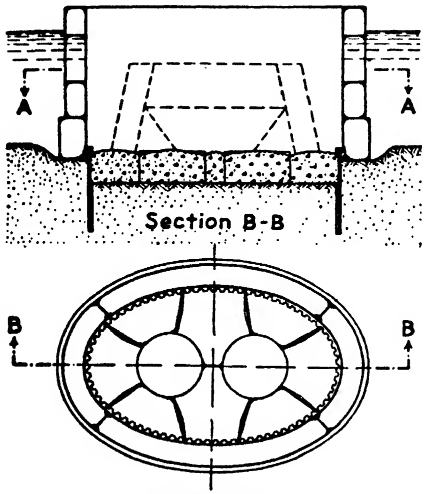


FIG. 11-21. Reusable cofferdam for piers of Denmark's Storstrom Bridge. These were used where ground conditions made construction under water necessary to prevent blowup of the bottom. Cofferdam is floated into place, sunk, and later refloated and moved to next pier site. *(See S. M. Koefoed, Four Cofferdams for Forty-one Foundations, Engineering News-Record, July 6, 1939.)*

portion of the pier was completed, the cofferdam was refloated and taken away, as shown in Fig. 11-22.

The chief modification used in Fig. 11-22 is the use of sheet piling driven outside the cofferdam, excavation in the dry, and the elimination of the tremie concrete. The seal between the piling and cofferdam was made by fitting wooden blocks into the troughs of the piling. When the cofferdam was unwatered, the pressure caused these to bear against a bulge around the outside bottom edge of the cofferdam. Divers then placed rope impregnated with tallow between the blocks and this projection. This proved to be sufficient. Jacks were built into the bottom of the cofferdam so that, when ready to float, they could lift the cofferdam vertically far enough to break away the bottom and beveled inside from the concrete.

11-8. Cellular cofferdams. Another type of cofferdam for large areas and high heads is the cellular construction pictured in Fig. 11-23. This is used mostly in connection with the construction of dams, locks, and other large water-front structures. There are various shapes and arrangements of cells, some being illustrated in the drawing.

The advantage of the circular cells is that, even though they are 40 to 60 ft. in diameter, the earth pressure on the inside causes only ring tension in the sheet-pile walls. These tensile forces are transmitted through the interlocks. This tension should ordinarily be limited to 12,000 lb. per lin.

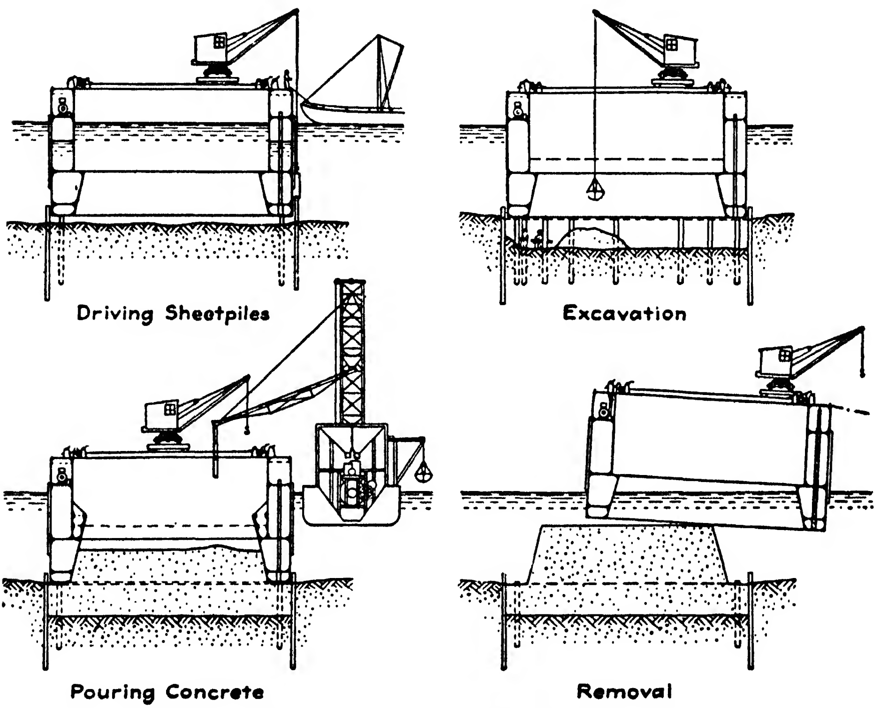


FIG. 11-22. Floating cofferdams for piers of Storstrom Bridge, Denmark. This shows the sequence of operations when impermeable ground permitted unwatering the area inside the cofferdam. (See S. M. Koefoed, Four Cofferdams for Forty-one Foundations, Engineering News-Record, July 6, 1939.)

in. of interlock. Their ultimate tensile strength is usually specified as a minimum of 16,000 lb. per lin. in. No wales or bracing are needed. When the unbalanced outside pressure tries to tip a cell over, as indicated in (*h*), the friction in the interlocks along the sides *AB* and *CD* offers considerable resistance to shear because it prevents the joints from sliding into positions like those in (*g*). If the interlocks are not in heavy tension, these frictional resistances may be small and unreliable. A few welds along the joints will remedy this, but they will cause extra trouble in dismantling the cofferdam.

This problem of providing for the shear in a large cellular cofferdam is an important one. Notice that, in Fig. 11-23(*b*), the cross walls are under tension even though the earth pressures against them counterbalance each other. This is because they tie the scalloped sides together. If this tension is not enough to cause sufficient friction, welding of joints or the attachment of diagonal bracing to these walls may then be necessary. Of course, the main shearing resistance is provided by the fill in the cells, hence it should be coarse sand, gravel, or stones.

Watertightness is also important. The sheet piling should provide fairly good tight joints for the walls. The leakage at the bottom can be

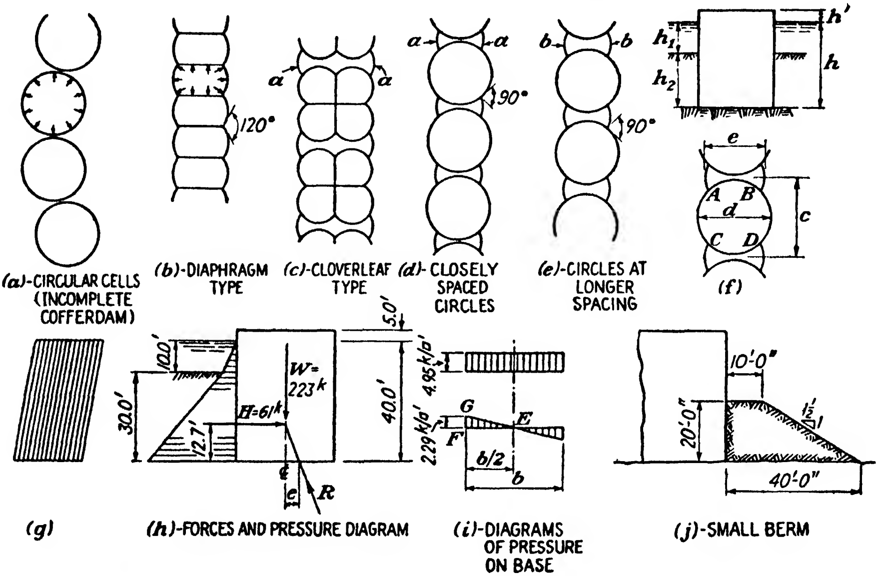


FIG. 11-23. Types of and miscellaneous material for cellular cofferdams.

resisted mainly by the sealing action of the fill on the rock or other bottom. In Fig. 11-23(*a*) it is apparent that leakage is probable between adjacent cells unless they are connected, and there is little chance to seal the bottom at the junction. The portions *a* and *b* in the other illustrations are primarily to prevent leakage rather than to provide structural resistance to overturning. From the standpoint of tightness, clay fill would be desirable; from that of strength, a coarse granular fill is better. Furthermore, the latter should be drained by providing weep holes near the bottoms of the inside piling. This adds greatly to the stiffness of a sand or gravel fill. Any berm used inside should be drained also. Such a berm is a great help. Of course, the greatest tensile stress in the interlocks would come when the fill in the cell is saturated but the inside of the cofferdam is dry. This eventuality should not be able to wreck the structure.

The stability of such a cofferdam should be entirely reliable and able to resist floods. The top should be covered with riprap or a 4- to 6-in. concrete slab to prevent scouring out of the fill in such an emergency.

Some details of junction pieces for piling[1] are shown in Fig. 11-25. The dimensions of these and of the cells will depend upon what type of sheet pile is adopted and how many pieces are to be used to construct a cell.

According to Hedman, the following data regarding coefficients of friction were found as the result of tests made for Kentucky Dam:

Gravel on rock	0.5
Gravel on steel	0.4
Steel on steel at interlocks	0.3

FIG. 11-24A. View inside the completed steel-pile cellular cofferdam erected by Dravo Corporation, Pittsburgh, for construction of the new navigational lock at Morgantown, West Virginia, on the Monongahela River. Water has been pumped out, rock is being removed, and the first concrete is being placed for the bases of the land and river walls. The project is under the supervision of the U.S. Army Corps of Engineers.

The clover-leaf type shown in Fig. 11-23(*c*) has the advantage of permitting the filling of cells more or less independently, whereas this cannot be done in the case of the diaphragm type in (*b*). Furthermore, if a diaphragm in the former should give way, the entire cofferdam might not fail completely, but that in (*b*) would do so.

A cellular cofferdam may be founded upon soil if the sheet piles can be driven down to reach material that is sufficiently impermeable yet able to resist the applied loads. The abutting power of the earth below the plane

[1] A. F. Hedman, Cofferdam Design for Kentucky Dam, *Engineering News-Record*, Jan. 1, 1942.

of excavation inside the cofferdam will resist horizontal shear and serve to counteract the bursting tendency of the embedded portion of the structure.

Example. A cellular cofferdam like that in Figs. 11-23(*d*) and (*f*) is proposed. The approximate diameter d is 50 ft.; the spacing c, 52 ft.; the width e of the closure pieces, 42 ft.; the height h, 40 ft.; the freeboard h', 5 ft. The cells are to be filled with 45 ft. of gravel and drained to the inside. The cofferdam is to rest on rock. It is hoped that no inside berm will be necessary. Analyze the cofferdam when h_1 is water 10 ft. deep and the soil outside is sand 30 ft. deep, with a mixture of gravel near the bottom. Will the structure be safe? A safety factor of 1.33 in all respects is desired. Assume that the angle of friction ϕ of the saturated sand is 28°; of moist gravel, 35°; also that the moist gravel weighs 110 p.c.f.; the saturated sand, 68 p.c.f. Use Rankine's formulas for earth pressures. Use 0.5 for coeffi-

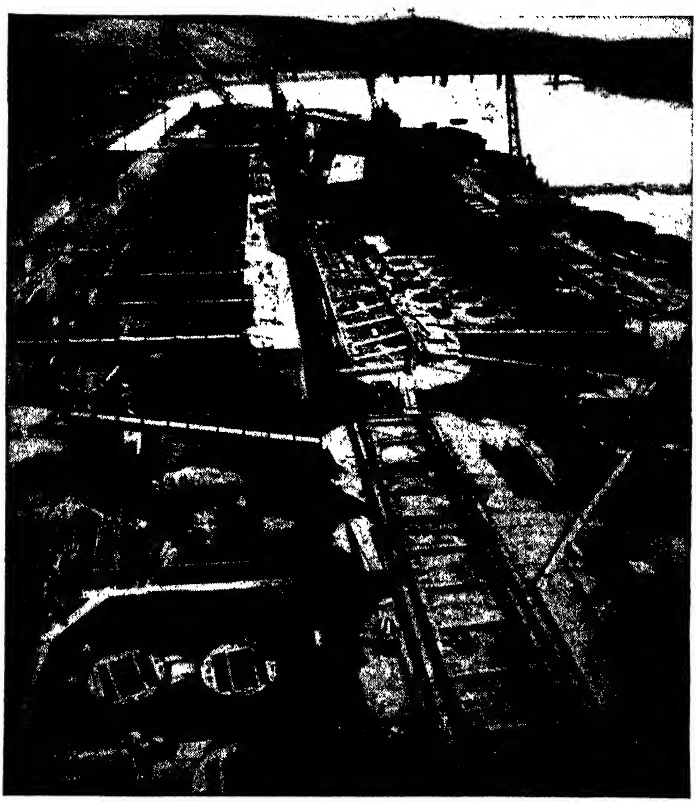


FIG. 11-24B. General view inside cofferdam as work progresses with reconstruction of Lock No. 2 on the Monongahela River at Braddock, Pa. The 1190-ft. long cofferdam encloses the area of the old riverward lock and also the new river and middle walls. The 36 main sheet-pile cells forming the cofferdam are about 55 ft. high from top down to rock. Contractor is Dravo Corporation, Pittsburgh, for the U.S. Army Corps of Engineers.

cient of friction of gravel on rock; 0.3 for steel on steel at interlocks. Neglect friction between the sand and the sheet piling.

It is satisfactory for this analysis to substitute a rectangular shape for the scalloped one, provided the areas are approximately equal. For the circular cell alone,

$$A = \pi \times 25^2 = 1{,}960 \text{ ft.}^2$$
$$\text{Width of square} = b = \sqrt{1{,}960} = 44.2 \text{ ft.}$$

Because of the extra area of the junction pieces, call $b = 45$ ft. The weight per foot of length of a typical unit, neglecting the weight of the piling is

$$W = 45 \times 0.110 \times 45 = 223 \text{ kips}$$

Lateral pressures on the outside for the diagram in Fig. 11-23(h):

$$\frac{1 - \sin 28^\circ}{1 + \sin 28^\circ} = 0.36$$

$$\text{Water} = 62.5 \times \frac{40^2}{2} = 50{,}000 \text{ lb.} = 50 \text{ kips}$$

$$\text{Sand} = \frac{68 \times 30^2}{2} (0.36) = 11{,}000 \text{ lb.} = 11 \text{ kips}$$

$$H = 50 + 11 = 61 \text{ kips}$$

Center of gravity for H:

$$(50 \times {}^{40}\!/_3 + 11 \times {}^{30}\!/_3) \div 61 = 12.7 \text{ ft. from bottom}$$

Stability:

Eccentricity of pressure at base,

$$e = 12.7 \times \frac{H}{W} = 12.7 \times \frac{61}{223} = 3.47 \text{ ft.}$$

This means that the resultant is well within the middle third of the base (as it should be) because $b/6 = 7.5$ ft. If e is multiplied by the safety factor, $e' = 1.33 \times 3.47 = 4.61$ ft., which is still safe. The cofferdam cannot tip over.

Pressure diagram in Fig. 11-23(i):

$$p = \frac{W}{A}\left(1 \pm \frac{6e}{b}\right) = \frac{223}{45}\left(1 \pm \frac{6 \times 3.47}{45}\right) = 4.95(1 \pm 0.463)$$
$$p_{max} = 7.25 \text{ k.s.f.} \qquad p_{min} = 2.66 \text{ k.s.f.}$$

Sliding:

$$H = 61 \text{ kips} \qquad fW = 0.5 \times 223 = 112 \text{ kips (safe)}$$

Stress on interlocks:

$$\text{For the gravel, } \frac{1 - \sin 35^\circ}{1 + \sin 35^\circ} = 0.27$$

Maximum pressure at bottom $= 110 \times 45 \times 0.27 = 1{,}340$ p.s.f.

Ring tension $= pr = 1{,}340 \times 25 = 33{,}500$ lb./ft. or 2,790 lb./in. (safe)

Longitudinal shearing resistance along vertical center line:

Total friction at interlocks in one wall,

$$H' = 0.110 \times \frac{45^2}{2} \times 0.27 = 30 \text{ kips}$$

$$\text{Ring tension} = H'r = 30 \times 25 = 750 \text{ kips total}$$

$$\text{Total friction} = 750 \times 0.3 = 225 \text{ kips for one side or wall}$$

$$\text{Average friction per foot of cofferdam} = \frac{225 \times 2}{52} = 8.6 \text{ kips}$$

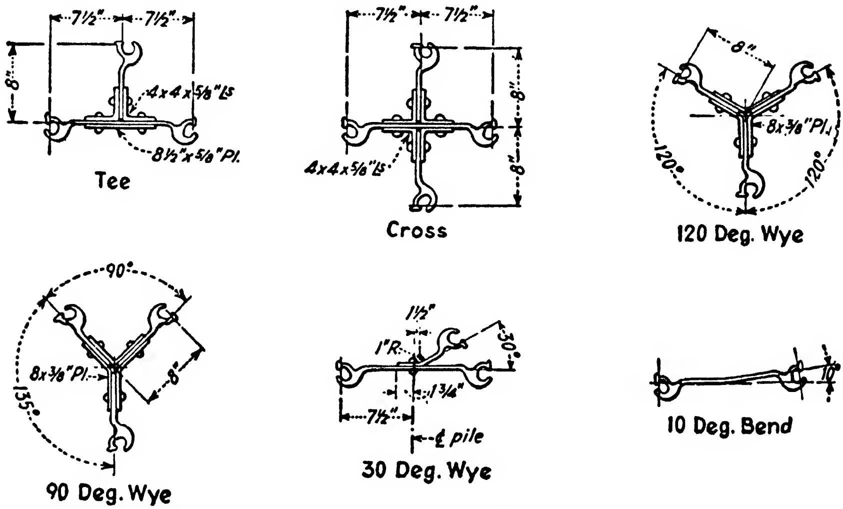


FIG. 11-25. Details of fabricated connections for steel sheet-pile cofferdams used for Kentucky Dam. (See A. F. Hedman, Cofferdam for Kentucky Dam, Engineering News-Record, Jan. 1, 1942.)

Shearing resistance of earth along center line, with $\tan \phi = \tan 35° = 0.7$,

$$S = H'f = 30 \times 0.7 = 21 \text{ kips per ft. of cofferdam wall}$$

Total resistance to vertical shearing,

$$S_v = 8.6 + 21 = 29.6 \text{ kips per ft. of cofferdam}$$

Force to be resisted and represented by triangle FGE, Fig. 11-23(i), is

$$V = 2.29 \times \frac{22.5}{2} = 25.8 \text{ kips}$$

The resistance S_v divided by V = S.F. = 29.6/25.8 = 1.15.

This is less than the 1.33 desired. Therefore, bracing may be attached to the sides to aid the interlocks, the interlocks may be welded near the top, the cells may be made larger, or a berm may be used on the inside. Probably the cells will not be completely and continuously drained. This would add to the ring tension and the friction at the interlocks, but the safety of the structure should not depend upon it. Furthermore, it is not desirable to depend too much upon the average value of the resistance to sliding at the interlocks when a structure is as big as this one.

If a small gravel berm is added as shown in Fig. 11-23(j), what effect will it have upon the longitudinal shear in the cofferdam?

Such a berm will offer considerable passive resistance if the cofferdam tries to slide or tip over. However, there is no danger in either respect in this case. It is difficult to see how the berm will offer any more resistance than it has to, this being the nature of passive resistances, but its use is good insurance. The weak part of the design is the shear along the vertical center line.

As a maximum, the berm might offer the full sliding resistance along the rock. This might be

$$\left(\frac{10 + 40}{2}\right) 20 \times 0.110 \times 0.5 = 27.5 \text{ kips}$$

Another resistance might be the passive pressure as an earth mass. If the top were level and wide enough, the abutting power might be

$$\frac{0.110 \times 20^2}{2}\left(\frac{1 + \sin 35°}{1 - \sin 35°}\right) = 22 \times \frac{1}{0.27} = 82 \text{ kips}$$

Obviously, this cannot be developed because the berm will slide first. Furthermore, the berm is not a level-top mass of earth of great extent.

The minimum effect of the berm will be its active pressure against the back of the cofferdam. This may be approximated by assuming a level top. Hence,

$$H_b = \frac{0.110 \times 20^2}{2} \times 0.27 = 5.9 \text{ kips}$$

The real effect of the berm may then be somewhere between approximately 6 and 27 kips.

Using 6 kips as a minimum, the counteracting moment will be

$$M_b = 6 \times \frac{20}{3} = 40 \text{ ft.-kips}$$

The resultant moment now is

$$M' = 61 \times 12.7 - 40 = 774 - 40 = 734 \text{ ft.-kips}$$

The new eccentricity is

$$e' = \frac{734}{223} = 3.29 \text{ ft.}$$

$$V' = \frac{223}{45}\left(1 + \frac{6 \times 3.29}{45}\right)\frac{22.5}{2} \quad \text{or} \quad \frac{M'}{0.667b} = \frac{734}{0.667 \times 45} = 24.5 \text{ kips}$$

The revised safety factor against shearing is now

$$\text{S.F.} = \frac{29.6}{24.5} = 1.21$$

Thus the berm alone does not improve the situation greatly unless movement of the cofferdam develops some of the passive resistance. Even then, such a small berm does not help this feature very much. Diagonal bracing added on the curved walls of the cells is not the most desirable thing; neither are long diagonal tie rods in such a wide structure. It would appear that the cells could be somewhat smaller in diameter, thus having more cross walls per foot of structure. If made deeper, the extra shearing resistance of the fill would be directly useful.

11-9. Miscellaneous practical features for cofferdams. A few of the many practical matters concerning the planning and building of cofferdams are given for the guidance of the reader. They will not be discussed in great detail. Each job has to be made to suit particular conditions.

1. Floods. These may raise havoc with cofferdams. Nevertheless, economy requires that there be some practicable limit to the height and freeboard of the structure. Floods that are likely to occur frequently during the months when the cofferdam will be in the building or use stage should be guarded against, and with some reserve to spare. Those floods that occur regularly but at an inapplicable season may be discounted if one can be sure that the useful life of the cofferdam will not extend close to that period. Major floods that come once in several years or decades generally may be discounted because one cannot afford to provide against such possible but improbable events. However, the establishment of the elevation of the top is a very serious matter.

2. Current. Because of its large area, the dynamic effect of flowing water may be a real force to consider in the design of a cofferdam. Records of stream and tidal currents should be searched for, and estimates should be made of the acceleration of velocity to be caused by the obstruction of the waterway when the cofferdam is in place. Scour is another effect that may be harmful. The pressure may be estimated by the formula

$$Ft = M(v - v_0) \tag{11-8}$$

where F = force in pounds, t = unit of time (taken as 1 sec.), $M = W/g$ = mass of water impinging upon the structure per second, v = original velocity in feet per second, and v_0 = final velocity in feet per second (usually taken as zero). The force is to be reduced to allow for any streamlining of the cofferdam that may be applicable.

3. Waves. Storms may cause waves that are hazardous for some cofferdams because of their large area and light weight. The trouble may come from flooding as well as from shock. This subject is discussed more fully in Chap. 13.

4. Unwatering. When a cofferdam is flooded, the interlocks of the piling may be loose because of the lack of pressure to close them. It may then be difficult to get centrifugal pumps of sufficient capacity to overcome the effects of so many leaky joints. After a head of several feet is acting upon the piling, the pressure is likely to close the joints and reduce the leakage in spite of the fact that the head increases as the unwatering proceeds. The movements of sheet piles produced by waves during the first part of the pumping may accentuate leakage and greatly interfere with securing of the initial lowering of the water. Dirt, sand, cinders, or similar materials piled around the cofferdam may help to "silt up" the interlocks. Another serious danger is the blowing up of the bottom during or after unwatering or during the driving of piles inside the cofferdam. Personal experience with the particular soils, or with similar ones, and the experiences of others are exceedingly helpful in estimating the head of water that can be withstood safely and without harmful seepage.

5. Unbalanced pressure. Cofferdams that have a considerable difference in the elevation of the earth on opposite sides, as in the case of a steeply sloping shore, must act as a sort of retaining wall. The resultant forces should be provided for, and the cofferdam should be studied as an entity. Small differential pressures may often be transmitted by bracing from the high to the low side, with the resistance provided by the passive pressure of the soil outside the latter.

6. Length of sheet piles. The ordinary maximum length of sheet piles is 80 ft. Some work has been done using welded extensions on standard piling, but this should be avoided in general.

7. Stresses in sheet piles. Sheet piles are very reliable material. Ordinarily the yield point is from 30,000 to 35,000 p.s.i.; the ultimate strength, 70,000 p.s.i. In such temporary structures, it might seem to be justifiable to use a high working unit stress. This, however, is not advisable generally because of the uncertainty about unit pressures, bracing reactions, and the stress conditions in the piling. The cost per pound of recoverable and reusable piling is so small compared to the cost of a failure that it is unwise to skimp on this particular item.

8. Investigation of soils. The soils at the site and the materials to be used for fills and berms should be studied carefully. Among the properties to be ascertained are the unit weights of soils wet, moist, and dry; the angle of internal friction for saturated, moist, and dry soils; the angle of repose; the compressive and shearing resistances; consolidation; and permeability.

9. Safety factor. Cofferdams are temporary structures. They do not need the reserve strength that is customarily required for permanent ones, yet they *must* not fail. When a cofferdam is studied carefully and when all reasonably probable loads are wisely estimated and provided for, the safety factor may be 1.25 to 1.5. When there are unpredictable elements to be withstood, the reserve may well be larger.

10. Reuse of materials. A cofferdam should be planned with proper consideration for its dismantling. So much material is generally involved that it is worth while to arrange the details of the structure so that the bracing can be taken apart and the piling pulled with a minimum of labor and waste.

11-10. Special methods of protecting excavations. The freezing process has been used successfully in some cases to form a sort of cofferdam around an area to be excavated. This is usually done by setting up a refrigeration plant, driving large pipes around the site, as shown in Fig. 11-26(*a*), inserting smaller pipes inside the large ones, connecting all into a closed system, and circulating cold brine down through the small pipes and up through the large ones. This causes the ground to freeze around the pipes and, if planned properly, the frozen regions around the adjacent pipes will eventually constitute a solid wall of frozen earth several feet thick. In certain cases, this may be strong enough to permit open excavation within the enclosure.

A few points to bear in mind regarding this freezing process are

1. The area of the excavation should not be too large.

2. The layout of pipes should be such as to constitute or approximate a circle in order to utilize the arch action of the frozen earth when the area is to be large. Small areas can be surrounded by square or rectangular layouts, but the frozen earth is not dependable when it has to act as a beam.

3. Water-bearing sand, gravel, and silt are suitable materials for freezing. Clay may be frozen, but other methods may be more desirable for such soils.

4. Conditions must be such that serious leakage in or blow-up of the bottom will not occur. The frozen region should therefore extend down to rock, hardpan, or dense clay unless the incoming water can be handled by pumps.

5. The freezing process is best suited to jobs of short duration, partly because of the cost of operating the plant and partly because the dependability of a frozen earth structure is not very good. Equipment or power may fail temporarily; floods or hot weather may cause conditions that exceed the capacity of the plant.

6. The process is particularly useful in protecting excavations in or at the foot of slopes where slides may be a menace. In this case, a portion of the slope is frozen to constitute a temporary dam to support the soil behind it. Such a situation is shown in Fig. 11-26(*b*).

Cement grout may be used to protect an area to be excavated if the soil is porous and the excavation is small. The difficulty is the inability to control where the grout goes. It may not form a complete and tight enclosure or cutoff wall. It can be used sometimes to assist in such special jobs as underpinning. It is primarily one method of consolidating the ground.

The uncertainty of control is illustrated by a case where grout was pumped down to a point some 20 ft. below ground. The operator realized that an unusual amount of high-pressure grout was being used. A helper soon discovered that it was coming to the surface as a boil many feet

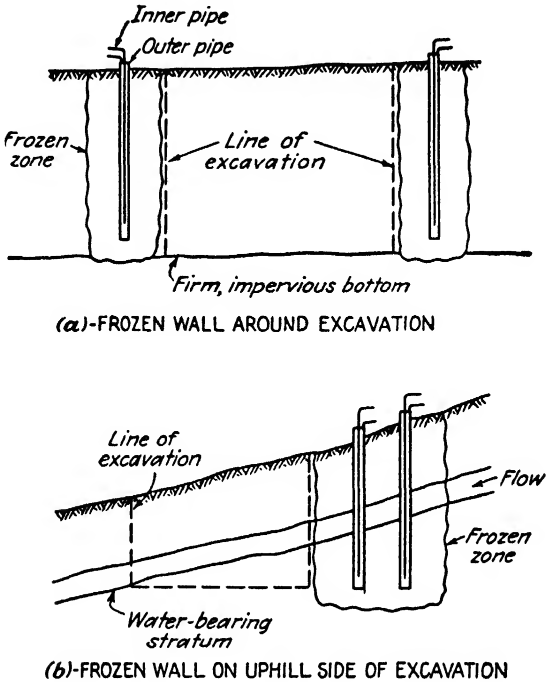


FIG. 11-26. Illustraton of use of frozen earth as protection for excavation.

from where it was introduced. It apparently followed a porous seam or stratum as the line of least resistance.

In planning grouting, one should remember that saturated sand cannot be grouted well without expelling the water already in the voids. Forcing grout into the sand rapidly may serve only to heave it and to form local seams or pockets of grout.

Chemical consolidation is another method that has limited usefulness. In general, with the Joosten method, one chemical is forced into the soil, then another one is added to cause the formation of solids or jells that stiffen the soil. This may be of some value, but cement grout should usually accomplish as good results and do so more economically.

The construction of the foundations of the floodlight towers at the Kezar Stadium in San Francisco's Golden Gate Park is an example of such consolidation methods applied to a local area.[1] A sodium silicate solution was pumped into the ground under a pressure of 200 p.s.i. to solidify the soil—a running sand fill—in the vicinity of the points where the bases of the pier columns were to be. This was 16 ft. below the surface. Working through 30-in. casings, the soil was excavated by hand to form belled bottoms some-

FIG. 11-27. Detail picture showing the sand filter around wellpoint riser pipes. This is part of the system for unwatering the site of the Neversink Dam Project in New York. The sand is placed around the wellpoint screen and riser pipe to aid in the drainage of fine soils and to prevent clogging the screens. This is an important feature in such soils as silt. Built by S. A. Healy Co., White Plains, N.Y. *(Courtesy of the Moretrench Corp., 90 West Street, New York City.)*

what like that of Fig. 9-7. The solidified sand developed a compressive strength of approximately 75 p.s.i.

11-11. Wellpoints. Much trouble in the making of excavations is caused by the presence of water, especially when the soil is fine sand or silt. When the water is allowed to flow into the excavation or to sumps, it is

[1] *Engineering News-Record,* Nov. 10, 1949, p. 48.

likely to cause movement of the fine particles, slumping of the sides, undermining of adjacent areas, or even softening and blowing up of the bottom. It may also cause a large increase in the amount of excavation because of the flat angle of repose of the water-bearing slopes. When the bottom is clay, water brought in by higher granular strata may cause serious muddying of the bottom.

In many cases, the use of wellpoints may be a means of lowering the watertable so that excavation can be made in moist soil that will stand on fairly steep slopes temporarily. A wellpoint system consists of a series of driven or jetted pipes, several feet apart, as shown in Fig. 11-27, with screened intakes. These pipes are attached to headers that are connected to centrifugal or other pumps which remove the water from the pipe line. The water runs through screens that are fine enough to keep out the particles of soil. The effect is somewhat as pictured in Fig. 11-28(*a*). The water flows in the direction of least resistance, to the screened intakes in this case. The result is the lowering of the watertable to some such surface as *ABCDE*, the slopes depending upon the permeability of the ground and the amount of water to be removed.

As an example, refer again to Fig. 11-28(*a*). The excavation to be made is indicated by the dotted lines *FGHJ*, and the soil is medium sand. The watertable is ordinarily at the level *KL*. If the pipes *a* and the wellpoints are driven to *B* and *D* and if pumping is carried on continuously, the water from the surrounding region will flow to and into the wellpoints until a balance between supply and pumping is secured. Then, if water comes from the sides only, the portion *BCD* of the watertable will become and remain rather flat because there is no renewal of its supply. If water also flows upward from below, *BCD* will be curved more sharply. The system should be planned so that the watertable is below the bottom *GH* of the excavation. After the structure is completed, the pumping is stopped and the wellpoint system is removed. The water will then return to its natural level.

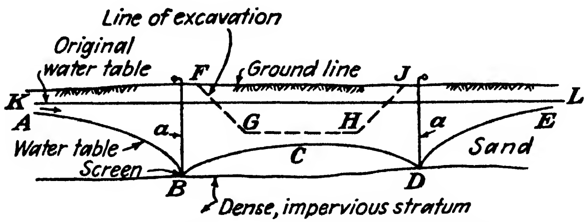
When planning a wellpoint system, one should study the permeability of the soils and the quantity of water to be handled. A pumping test may be very helpful in judging whether the ground will drain properly and in estimating the capacity of the pumps required. The advice of those who make and use wellpoint equipment should often be secured before one starts a major installation of this character.

Some points to bear in mind regarding the installation and action of wellpoints are the following, referring to Fig. 11-28:

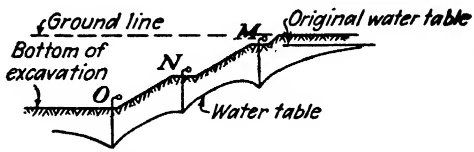
1. Suction pumps cannot lift water much above 20 ft. Therefore, a practical limit should be set as 15 or 16 ft.

2. When greater lifts are necessary, deep-well pumps may be used. These are lowered into the holes and can push the water to substantial heights. However, this installation is expensive.

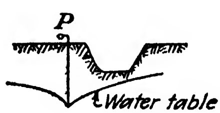
3. As shown in Sketch (*b*), it is sometimes practicable to make a large excavation in stages. A row of wellpoints is placed around the site at position M, the watertable is lowered, and the first portion of the excavation is made to N. Another row of wellpoints is then added at N, and the



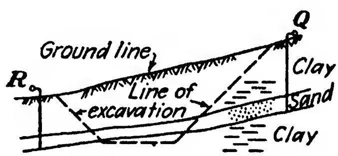
(*a*)-ACTION OF WELLPOINTS AROUND AN EXCAVATION



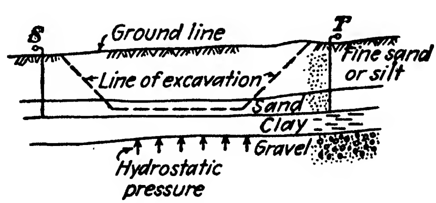
(*b*)-WELLPOINTS INSTALLED IN STAGES FOR DEEP EXCAVATION



(*c*)-TRENCH EXCAVATION



(*d*)-DRAINING OF A WATER-BEARING STRATUM



(*e*)-DANGER OF BLOW-UP OF BOTTOM

FIG. 11-28. Use of wellpoints in aiding open excavations.

work is continued. A third row may then be placed, etc. At the bottom, a final row like that at O may be necessary to keep the bottom sufficiently dry.

4. The ground around shallow trenches and other narrow excavations may be drained sufficiently by the use of one row of wellpoints P, as shown in (*c*).

5. Excavations on hillsides may be difficult to make because of one or more porous water-bearing strata, as shown in (*d*). One row of wellpoints at *Q* may be able to intercept the water. If the excavation is near the base of the slope, wellpoints *R* may also be necessary because of back pressure on that side.

6. Sketch (*e*) shows a condition where wellpoints *S* and *T* may drain the sand layer satisfactorily. However, hydrostatic pressure in the gravel stratum below the clay may heave up the bottom. Deep borings should be

FIG. 11-29. A Moretrench wellpoint system for unwatering the excavation for a pumping station for the city of Muskegon Heights, Muskegon, Michigan. This shows the header pipes at different levels. Notice how close the excavation is to the lake. (*Courtesy of the Moretrench Corp.*)

made to reveal such conditions, and especially to see if there is likely to be artesianal action due to water under high pressure trapped in a porous layer under an impervious one. Deep jetted holes, sometimes called deep wells, may be used to tap such a layer and to relieve the pressure, or they may prove to be like leaks in a water main, that cannot be stopped without great difficulty.

7. Very fine sands and silts tend to hold water by capillary action, and they offer considerable resistance to percolation. Wellpoints in them may be ineffective unless they are very close together. The vacuum method[1] applied through a wellpoint system may then be of value.

[1] Terzaghi and Peck, "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc.

8. Clays cannot be drained by wellpoints because of their impermeability.

9. In deep excavations, made as in (*b*), there may be danger of instability of the slopes because the unwatered soil is only a relatively thin layer along the surface with water under pressure behind it.

10. If too great a quantity of water has to be pumped, the flow through the soil may remove or disturb too much of the soil itself.

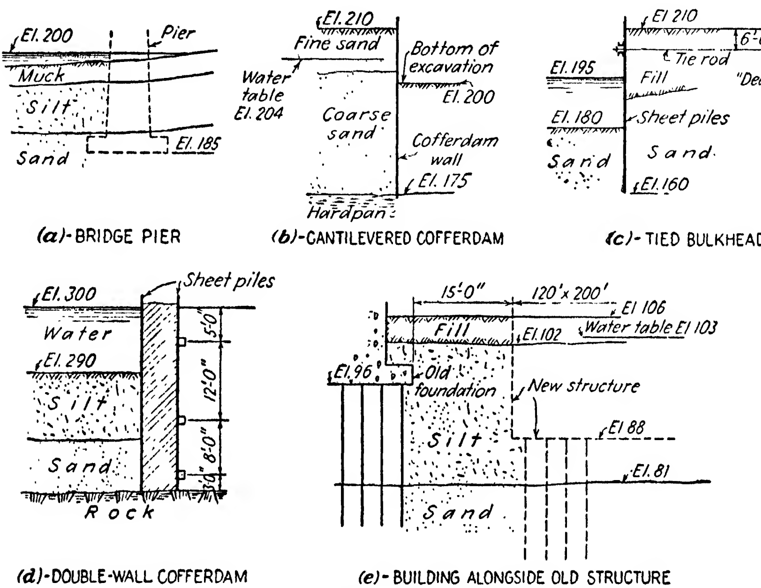


FIG. 11-30. Miscellaneous data for problems.

11. Once having started the pumping of wellpoints, it should be kept up continuously. Hence, provisions should be made for breakdown of equipment or power.

An interesting example of the use of wellpoints occurred recently. An institutional building was to be erected on a site that the new owners inspected "while driving by in their automobile." They saw that a swale extended diagonally across it and that skunk cabbage and similar plants grew there, but they made no attempt to investigate conditions. After the contract was let, the contractor started digging for footings. He found that, just under the mucky top, the soil was a very fine saturated sand that flowed almost like thick gravy. He managed to get down a few feet to hardpan and to build two of the footings. As the runny sand was much deeper at the next position, he thought that he would dig a sump behind the structure, install pumps in it, and drain the ground that way. He started to do this and connected the pumps to a suction pipe. As he lowered the water level a little, the sand ran in. He was soon pumping sand and water and ruining the pumps. Furthermore, the sump was soon as big as a house. He finally gave up and hired someone else to install wellpoints. This was soon done, the area was kept free of excess water even though the 6-in. header was running full of clear water. It was then revealed that the water-bearing sand was a few feet deep at one end of the building and 22 ft. deep at the other. The footings were placed on hardpan inside deep pits, piers were extended to the first-floor level, and the floor was made a self-supporting reinforced-concrete structure. Of course, the owners "paid through the nose" for all this extra work.

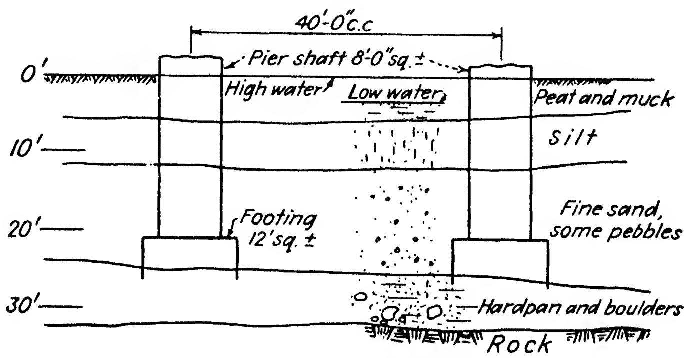


FIG. 11-31. Problem in planning construction of a viaduct pier in a swamp.

The use of wellpoints may lead to unwise conclusions if the bearing value of a soil is judged by its action during excavation after the soil has been "dried out." Allowable pressures and depths of foundations should be determined by tests and studies of the soil in its normal condition. The firming action of capillary moisture retained in the earth after unwatering may cause a weak, fine, runny sand to appear to be strong whereas it is weak when saturated. One example of disastrous results occurred recently. No borings had been made at the site of a group of school buildings —almost an unpardonable neglect. When excavation was started, underground water and springs were encountered. Wellpoints were installed at extra cost to the owner. When the excess water was removed and kept down, the excavation was continued. The fine sand seemed to be firm, and the footings were built as planned. Now, a few months after completion of the buildings, unsightly cracks are appearing, and they are obviously the result of uneven and considerable settlement.

PROBLEMS

11-1. A single-braced single-wall sheet-pile cofferdam is to be constructed around the base of a bridge pier. The soil profile is shown in Fig. 11-30(a). Design the cofferdam.

Suggestions: Do not overlook the fact that the pier is to rest on sand. Assuming the buoyant weight of the sand to be 70 p.c.f., the piling should extend down at least far enough to have this weight balance the hydrostatic head of 15 ft. Perhaps it should extend much farther.

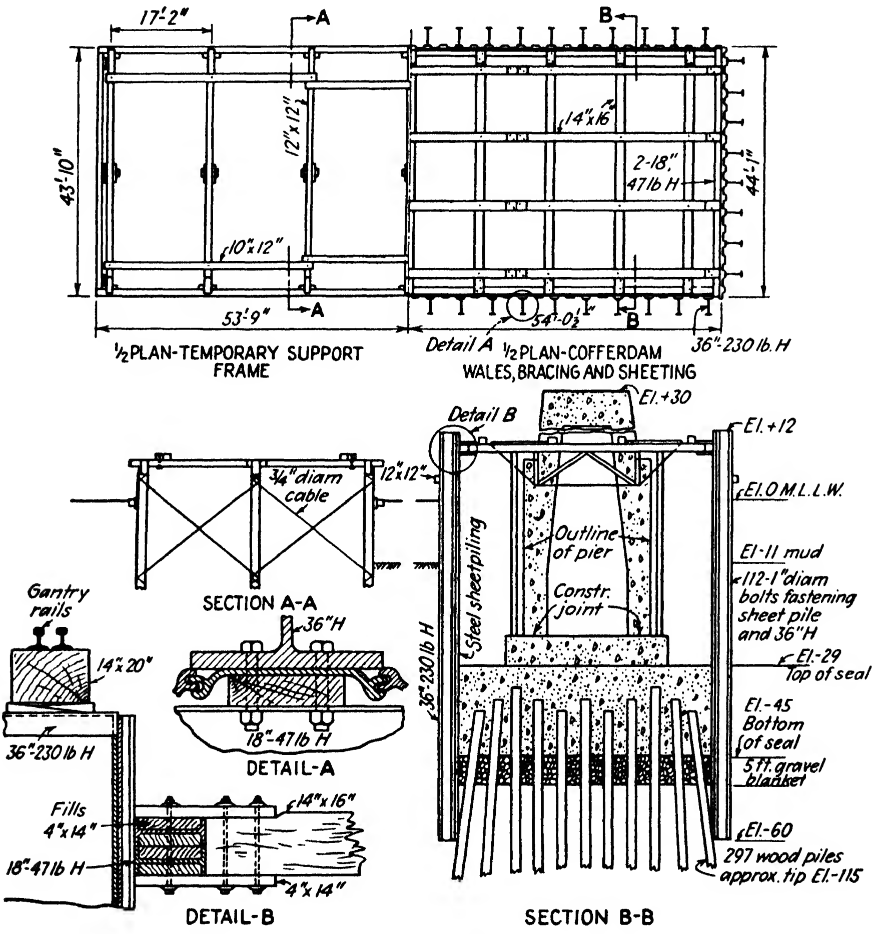


FIG. 11-32. Construction of a pier on piles inside a cofferdam using steel sheet piling with heavy steel beams as master piles to reduce interior timber bracing. *(See Purcell, Andrew, and Woodruff, Deep Open Caissons for Bay Bridge, Engineering News-Record, Aug. 23, 1934.)*

11-2. A cantilevered single-wall cofferdam is to be built around a large excavation. The piling goes through a saturated top layer of silt 12 ft. deep. Under the silt is a deep layer of fairly dense clay. The bottom of the excavation is to be 2 ft. below the top of the clay. It is proposed to drive the sheet piles 24 ft. below the bottom of the excavation. Is this adequate? If so, estimate the maximum bending moment in the piling.

11-3. Analyze the single-wall cantilevered cofferdam pictured in Fig. 11-30(*b*). Will it be safe? Estimate the bending moment in the piling.

11-4. Estimate the maximum bending moment in the sheet piles of the bulkhead shown in Fig. 11-30(*c*), the bending moment in the wales as simply supported beams, and the tension in the tie rods. Is the embedment safe? Ties are 8′ c.c.

11-5. Assume a double-wall multibraced cofferdam, as shown in Fig. 11-30(*d*). Estimate the maximum bending moment in the inner sheet piling as though it acted alone. Estimate the pressure per foot on each of the wales.

11-6. Assume the same cofferdam as in Fig. 11-30(*d*) except that the wales are located as follows: one at El. 300, one at El. 290, one at El. 280, and one at El. 274. Estimate the pressure on the wales and the bending moments in the piling, assuming the inner row to take the load.

11-7. Assume a diaphragm type of cellular cofferdam like that in Fig. 11-23(*b*). Assume a radius of curvature of 30 ft., a maximum width of cell of 70 ft. (10 ft. between centers of the two circles), a spacing of diaphragms of 30 ft., a depth of excavation of 32 ft. to rock, and saturated fine sand outside the cofferdam with $\phi = 30°$. Test its safety. The fill is gravel, with $\phi = 33°$. No berm is used inside.

11-8. Assume a cellular cofferdam like that of Fig. 11-23(*f*). The diameter is 60 ft.; the spacing, 64 ft.; the width of closure sections, 50 ft.; the depth of excavation, 35 ft.; the soil outside, 35 ft. of fine, saturated sand ($\phi = 30°$); the fill, 40 ft. of gravel ($\phi = 33°$). Is the cofferdam safe?

11-9. A large new building shown by the dotted lines in Fig. 11-30(*e*) is to be founded upon cast-in-place concrete piles under a mat. It is to be alongside the existing building shown at the left, and a street is 20 ft. from the side. How would you conduct the work so that the piles can be driven in the dry, and the building and street be protected?

11-10. A viaduct pier is to be built, as shown in Fig. 11-31. How can the work be done to best advantage?

12
CAISSONS

12-1. Introduction. The term *caisson* denotes a structure that is used as a protection and aid in the making of excavations for foundations but remains in place as a part of the permanent structure. In general, a caisson is sunk into position as the excavation proceeds inside of or under it.

Caissons may be large or small, deep or shallow. Large ones are especially useful for heavy bridge work; small ones, for buildings and miscellaneous structures. They are exceedingly useful in the making of deep foundations where other methods are inadequate or undesirable. Their use in shallow excavations is generally limited to cases where some special conditions make them necessary.

The design, construction, and sinking of caissons are parts of engineering in which much is to be gained by experience. Many things are done in certain ways to meet certain conditions because they have been found to be successful in the past. Some of the work involves theory but much is practical engineering, and it requires "know-how."

In this chapter an attempt is made to discuss the kinds of caissons that have been used, to show the general principles underlying the planning of them, and to illustrate some of the conditions for which each type is best suited. Much will be covered by drawings of various arrangements that have been used. By necessity these drawings have been simplified considerably in order to show the basic ideas without including too much detail. A collection of the working drawings giving all the details of just the commonly used types would constitute a book in itself. This cannot be attempted here. If it were, the expert would not need to look at it, and the beginner would be lost in a mass of material because he would not understand the reasons behind it all.

12-2. Open caissons. As its name implies, an open caisson is one that has no top or bottom during its use as a protection for the excavation process. It is like a box with vertical sides only, a honeycomb structure with open ends. In plan, it may be square, rectangular, circular, oval, or whatever shape best suits the situation. If small enough, the center may consist of one opening or well; if large, it may be subdivided by vertical partitions into a series or group of wells.

Figure 12-1 illustrates a very simple open caisson and the operations of sinking it. This is for the foundation of a tall approach pier that is to be part of a high-level bridge. It is near the shore line where the muck has

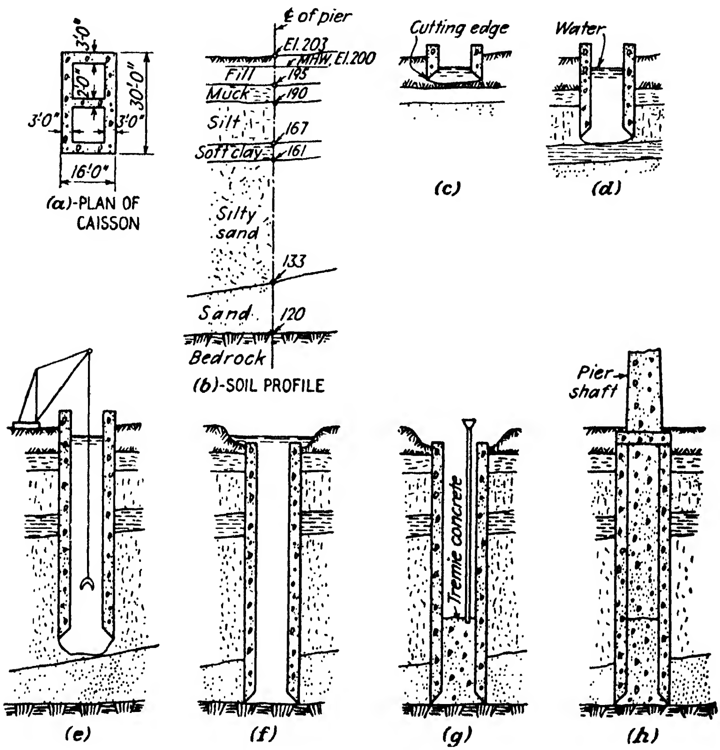


FIG. 12-1. Illustration of sinking of an open caisson.

been covered with a blanket of fill. The structure of the caisson is of reinforced concrete.

The following paragraphs refer to the sketches in Fig. 12-1:

(*a*) This is a plan of the caisson. Assume tentatively that the dimensions are as shown.

(*b*) This gives the soil profile at the center line of the pier. The pier is to be founded upon the rock at approximately El. 120. The datum is chosen so as to avoid any minus elevations because their use is likely to cause errors in the office and fieldwork.

(*c*) It is not so difficult to start a caisson when there is ground at the surface to work on, as there is here. A small excavation is dug at the surface, then the cutting-edge portion of the caisson and a small section above it are built in the position where the pier is to be. In this picture, some excavation under water has been carried on, and the caisson has started its downward course.

(*d*) It sounds simple to say that the further sinking of the caisson is accomplished by excavating under it and letting it settle into the hole because of its great weight. This is substantially what is done. However, one can realize that it is difficult to control the excavation because it has to be made by equipment working under water and within the limits of the area that can be reached by clamshell buckets (or other devices) lowered through the open areaway. If one side slumps in from under the cutting edge but the other does not, there is a tendency for the shaft to hang up on the latter side. If this persists, it may cause tipping of the caisson. Once tipping has occurred, it is difficult to correct. Adding weight to the high side and trying to undermine the higher cutting edge may cause the caisson to straighten up as it sinks farther. This is obviously an uncertain control. It is therefore very important to have the caisson started in the right location and kept in a vertical position during this early stage of the sinking process.

(*e*) The caisson is now shown in a position well advanced toward the rock. By this time the supporting effect of so much earth around it will tend to compel it to continue its course. This is fine if the sinking has been done accurately thus far; it is correspondingly difficult to correct any errors in its position. Friction of the soil bearing against the outside may now constitute very real resistance to further sinking. Pipes for water jets should be built into the caisson so that the jets can be used to loosen the material next to the caisson and reduce its frictional resistance.

(*f*) Here the caisson is shown with its cutting edge on the rock. The last several feet of progress may be extremely difficult when boulders, hardpan, or irregular rock is encountered. When a cutting edge is hung up on a large embedded boulder that is under one side, there really is trouble on hand. What can be done with it when it lies from 100 to 200 ft. below the surface and cannot be reached by the buckets or dislodged by water jets? If not too deep, divers may be able to undermine it or blast it out. If too deep for that method, small blasts set off near it may do some good, or they may do harm to the caisson or the surrounding areas. At best, they are a dangerous stab in the dark. Hardpan may also be difficult to remove from under the portion of the caisson that is shielded by the latter's thickness. A cutting edge can cut into soft soil, but it is not a pickax to chop away resisting materials. Compacted sand or gravel under one side and running sand under the opposite one may also create a difficult situation. When the rock is irregular or steeply sloping, it is usually very hard to excavate the rock so as to seat the caisson properly. In addition, disintegrated and seamy rock may not be removed and cleaned properly to secure adequate and even bearing. This phase of work—seating the caisson and cleaning the bottom—is a very important one.

(*g*) In this view, the inside of the caisson has been filled with a deep plug of tremie concrete, and the water above it removed. This latter may not be necessary since, if tremie concrete is good enough for the lower part, it should be sufficient for the upper portion of the fill or body also. Notice that the top of the caisson is below the ground level so that a sort of footing of the pier can be extended over the walls and the concrete fill without projecting above the surface. In some cases, it may not be necessary to use more than the tremie-concrete plug in the bottom, but solid concrete seems to be desirable here.

(*h*) This indicates the completed structure with the pier in place. The caisson is now merely an integral part of the heavy solid shaft that supports the bridge.

Open caissons may differ greatly in shape, material, and details but the preceding paragraphs show the general principles involved. The caissons

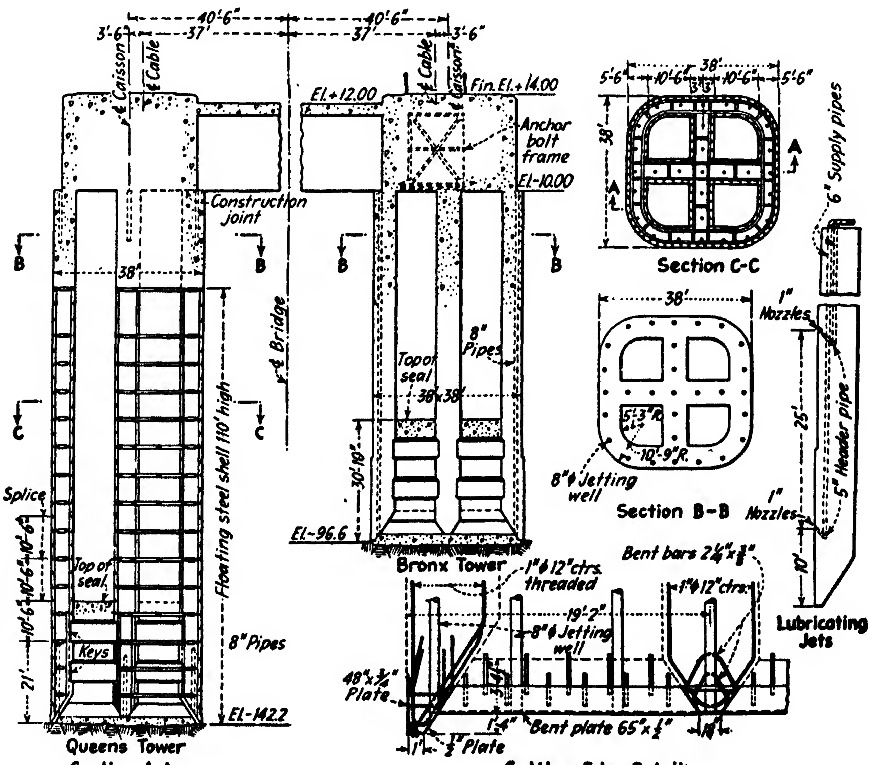


FIG. 12-2. Details of construction of caissons under towers of the Whitestone Bridge. (See George L. Freeman, *Deep Caisson Work at the Whitestone Bridge, Engineering News-Record, Aug. 3, 1939.*)

for the towers of the Bronx-Whitestone Bridge at New York are pictured in Fig. 12-2. The Queens anchorage for this bridge consists of four caissons. The riverward pair were 33 by 100 ft. in plan, were built of reinforced concrete with 5-ft. walls and partitions, and had steel cutting edges. They were sunk approximately 150 ft. below water. The rear pair of caissons were 24-ft.-diameter cylinders of reinforced concrete with a 6-ft. wall. These were also sunk approximately 150 ft. In all four cases, excavation was carried several feet below the cutting edge because of sloping rock. The bottoms of the cells of all caissons were sealed with concrete plugs extending 40 ft. above the cutting edges in some cases and full height in others. These cases show how varied the shapes and uses of caissons may be.

12-3. Pneumatic caissons. A pneumatic caisson is a caisson with a permanent or temporary bottom in it so arranged that men can work in the compressed air trapped under it. In this way, the difficulties described in connection with Fig. 12-1 (*f*) can be overcome. On the other hand, the work may be much slower, and it almost certainly will be more expensive. Nevertheless, its successful completion and the safety of the structure may require the use of compressed air.

When the borings show the likelihood of serious and perhaps hazardous difficulties for the sinking of an open caisson, a pneumatic one should be used, or an open caisson may be made with provisions for the installation of compressed-air work if it becomes necessary.

Figure 12-3 illustrates the principles of a pneumatic caisson and the operations in the sinking of one. This caisson is now supporting the New York River Ventilation Building of the Lincoln Tunnel. The sketches are greatly simplified. The following comments refer to the corresponding drawings:

(*a*) The caisson is composed of an inner and outer skin plate of steel, with steel trusses or girders as horizontal supports to form a boxlike structure. The cutting edge is fabricated steelwork with strong triangular brackets to support it. A working chamber, approximately 10 ft. deep, is made by installing an airtight roof of heavy beams and buckle plates as shown. Access to the working chamber is secured through the tubular shafts and air locks, which will be described later. In this case, the lower portion of the approximately 40- by 50-ft. caisson was fabricated as a unit, floated to the site, and anchored. The sinking was started by filling the spaces between the skin plates with concrete after the steelwork was built high enough to maintain freeboard.

(*b*) By maintaining compressed air in the working chamber at a pressure sufficient to counteract that of the silt and water at the cutting edge, men could excavate the silt from underneath the caisson. This was dirty laborious work. When a few feet of silt had been excavated, the caisson was usually sunk by adding more steel and concrete, and when the men had left the working chamber, by temporarily decreasing the air pressure. Rapid release of air pressure under the caisson is relatively equivalent to giving it a downward shove. This process was repeated many times as the caisson progressed downward.

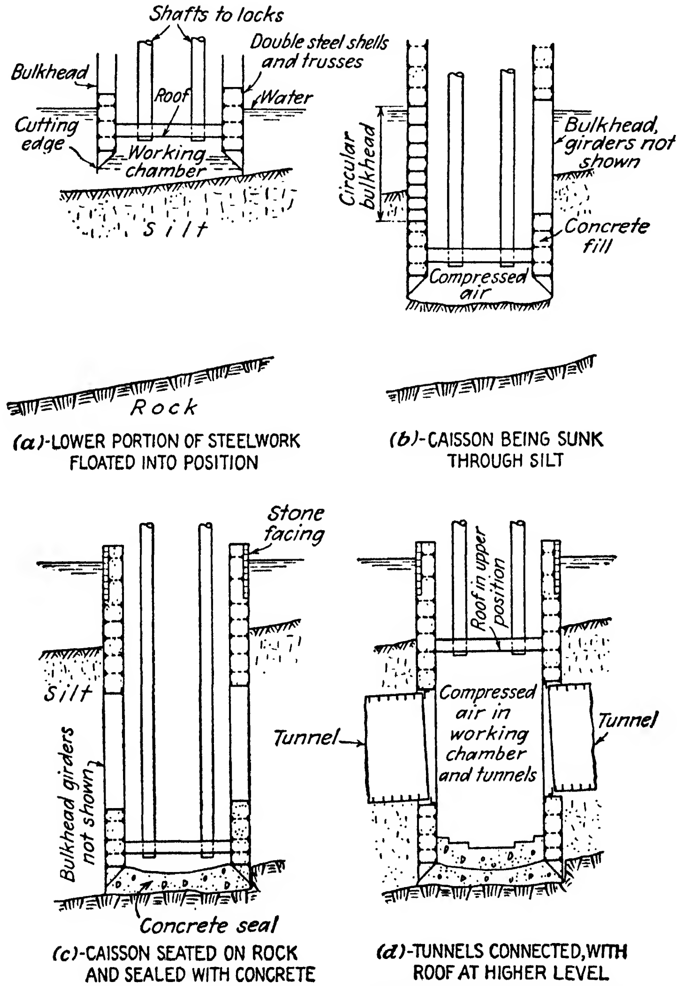


FIG. 12-3. New York river shaft of the Lincoln Tunnel built by the Port of New York Authority under the Hudson River at New York City.

(c) When the caisson was close to the rock, the latter had to be cleaned off and excavated to provide a firm level support for the concrete seal or invert that was placed under compressed air. This work was carried on about 90 ft. below the surface and in an air pressure of approximately 45 p.s.i.

An ordinarily limiting depth below water for compressed-air work is 100 to 110 ft.

(*d*) Circular bulkheads of steel plates and girders had been provided in two opposite walls of the caisson. After the latter was seated and sealed, it was necessary to extend the working chamber so as to have the roof above these bulkheads. The work in the tunnel shields was also done under compressed air. They approached from opposite sides until they were close to the bulkheads, then the latter were burned out, the tunnels advanced, and the tunnel linings joined to the caisson walls as indicated here. After that the compressed air was released, the roof removed, and other construction carried on. The caisson now made "an open hole in the river."

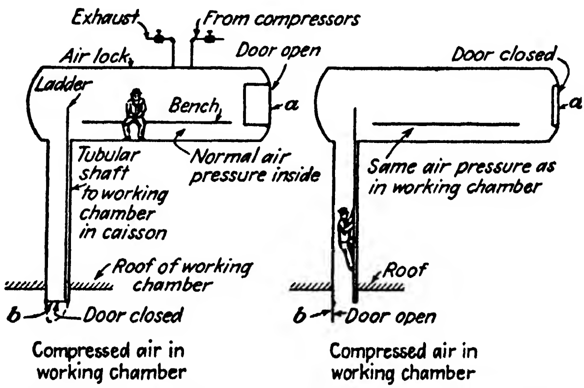


FIG. 12-4. Simplified illustration of operation of air lock for access to working chamber of a pneumatic caisson.

Compressed-air work is rigidly controlled by law as far as working time, maximum pressures, and, generally, wages are concerned. It is hazardous work, but it can be done without serious consequences if planned and executed properly. A large item of cost is for manual labor when pressures are high. For example, the regulations affecting the work on the caisson of Fig. 12-3 permitted a man to work a normal 8-hr. shift with a lunch period when the air pressure was at or below 18 p.s.i. For higher pressures, the working time was decreased. For example, under a pressure of about 45 p.s.i., a man could work 45 min., then he had to lock out and rest for approximately 5 hrs. After that he could go back and work for another 45 min. Not only was a man paid his regular daily wage when he worked this shorter time under high air pressure but he was paid a bonus also.

The general principles of the operation of the air locks is illustrated by Fig. 12-4. In Sketch (*a*), a man who wants to enter the working chamber has gone into the man lock through door *a*. Door *b* is kept closed so that the compressed air in the working chamber cannot escape. The man sits down while a lock tender closes door *a*, turns on the air pressure, and lets the pressure in the lock build up to equal that in the chamber. The door *b* is then opened, and the man descends to the working chamber, as shown in (*b*).

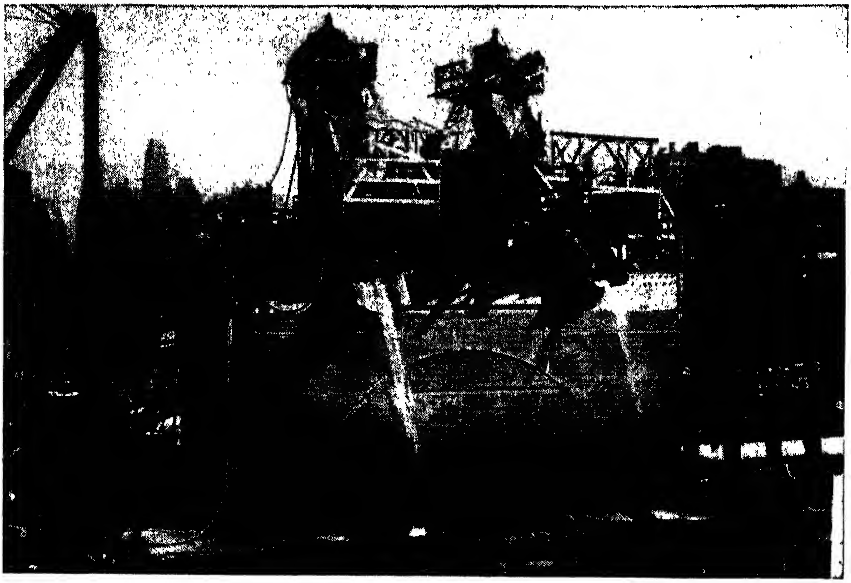
When leaving, the procedure is reversed. Door *a* remains closed while *b* is open. The man climbs through the shaft to the lock and sits down, door *b* is closed, the exhaust valve is opened, the air pressure drops slowly until the lock has normal pressure like that outside, door *a* is now opened, and the man goes out. Going under air pressure can be accomplished fairly rapidly. Coming out is a slow and chilling process, but slowness is necessary in order to allow the body time enough to eliminate the excess air absorbed or taken in when under pressure. Too rapid a decompression may cause bubbles of air to be trapped in muscles, joints, or the blood, producing caisson disease or "the bends." For example, the law may require that the rate of decompression be 1 min. for each pound of pressure up to 30 p.s.i. and 1½ min. for each pound in excess of that, because this rate has been found to be successful in most cases. Rigid control and observance of regulations will almost always prevent serious trouble.

The handling of materials through the locks is similar to that for the men except that it can be much quicker when no men accompany the materials. It is, of course, a slower and more costly process than when the operations are performed in open air.

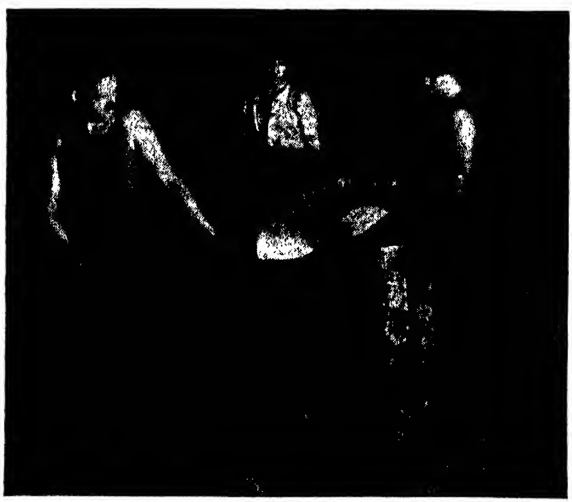
The material locks (and occasionally the man locks) used in caisson work are generally vertical cylinders. They are provided with a trap door at the bottom, means for passing buckets and cables, and a side door for the discharge of material dumped from the buckets. Such a lock is shown in Figs. 12-5 and 12-23.

When it is so expensive, why could not the caisson in Fig. 12-3 be sunk as an open caisson? The chief reason is the necessity of making sure that it can be sunk without insurmountable difficulties, and that it can be seated and sealed properly because it is to support a 150-ft. building that will house ventilation equipment. Old piles, riprap, sunken timbers, forgotten bulkheads, and miscellaneous debris are among the things that may cause trouble when working at old water fronts. Furthermore, the rock was known to be sloping, it was essential to get a level bearing, to remove disintegrated rock and any unexpected boulders or knobs, to clean out any crevices, and to have the bottom thoroughly concreted.

Pneumatic caissons may be used for purposes other than foundations and sometimes are sunk more than 100 or 110 ft. below ground. One such case, illustrated in Fig. 12-6, shows one of two exploratory caissons that were sunk as part of the core wall of the Merriman Dam of the Delaware water supply for New York City. Since such caissons had been used for the cutoff wall



(a) The caisson being sunk.



(b) A view inside of the working chamber.

FIG. 12-5. Views of the caisson for the New York River Ventilation Building of the Lincoln Tunnel. *(Courtesy of the Port of New York Authority.)*

of the Quabbin reservoir, it was desired to learn whether or not similar construction would be suitable in this case where the ground varied from runny sand or silt to glacial till and boulders, and most of it was water-bearing.

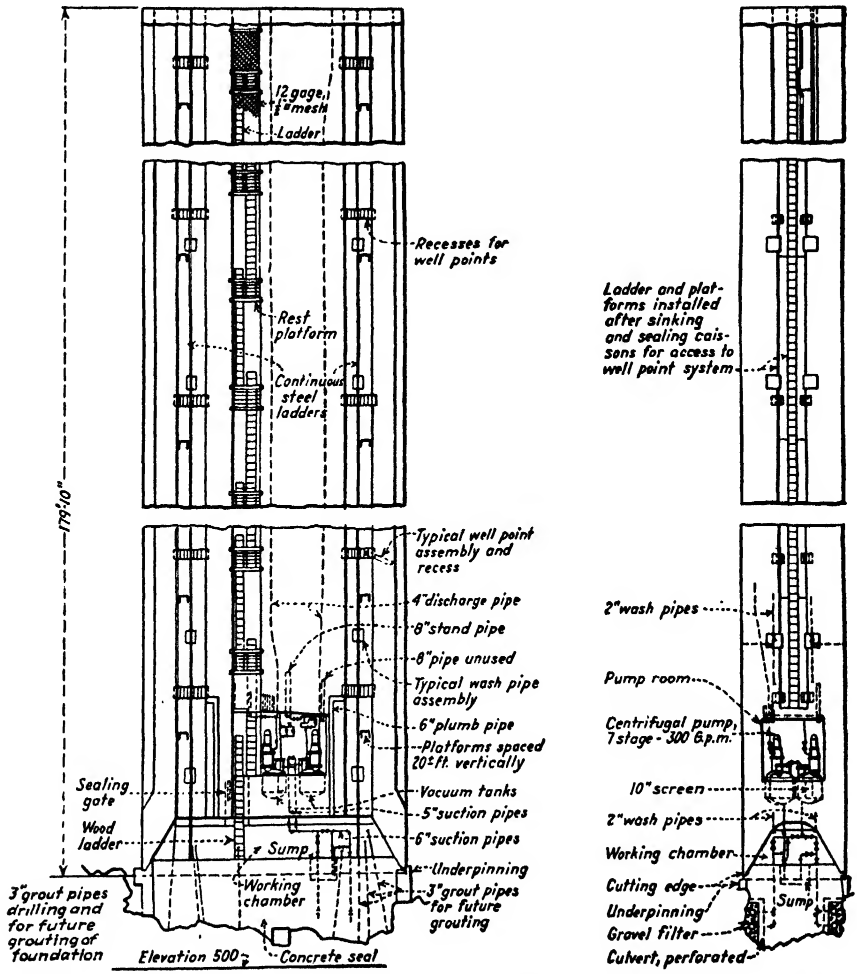


FIG. 12-6. Two pneumatic caissons like this were sunk through 180 ft. of alternating water-bearing and impervious materials at the Merriman Dam site. The air pressure was kept to a maximum of 32 p.s.i. by lowering the ground water. (*See Caissons for a Cutoff Wall, Engineering News-Record,* Dec. 5, 1940.)

There was artesian action in the upper seamy part of the rock and in some of the soil.

The caissons were reinforced concrete 15 by 38 ft. in plan, and ultimately 180 ft. high. They were located with a clear space of 48 ft. between them, They were started in a 17-ft. trench, sunk about 35 ft. in free air, then continued under compressed air except when in impervious ground or that with little water. As shown in Fig. 12-6, the caissons were equipped with pumps to lower the ground-water level, operating like wellpoints. This equipment was able to lower the watertable about 120 ft. so that the caissons were sealed under an air pressure of only approximately 26 p.s.i. Special gravel filters and perforated pipes were installed for future pumping.

Jet pipes were installed in these caissons but were not used. No top load had to be applied although, when under air, the caisson had to be "blown" to start it downward. Estimates of sliding skin friction with the caisson moving gave values from 570 to 924 p.s.f.

12-4. Materials and details. Caissons are generally made of steel, reinforced concrete, or a combination of the two. This is natural because they are needed where excavations are deep and pressures are heavy. Wood may be used to some extent. The structures must be tight, generally must be built in successive lifts as the sinking progresses, should be durable, and should be heavy for aid in sinking. These are things for which steel and concrete are particularly adapted. In many cases, as in Fig. 12-1, the interior is filled with concrete so that the caisson is not heavily loaded in the finished substructure. In others like Fig. 12-3, the caisson itself is the final substructure and must support large loads.

The choice of steel or concrete depends upon many things. The former can be fabricated in advance and erected quickly, and boxlike construction eliminates the need for forms. It is also strong and ductile. It will withstand great inequalities of loads without collapse, as when a cutting edge strikes an obstruction. It can be planned so as to provide for attachments of many kinds, such as the provision of connections for the roof of a working chamber in an open caisson if the need for compressed-air work arises. As in Fig. 12-3(*a*), a large portion of the caisson can be floated into place when there is sufficient depth of water. The steelwork may be riveted and calked for watertightness, or it may be welded. When the caisson must be started in shallow water and muck, it is an advantage to be able to assemble the steel shell by working from temporary platforms, to lower the steelwork alone until it is self-supporting or is borne by the muck, and to add the heavy concrete fill when it is convenient to do so.

On the other hand, reinforced concrete alone is sufficient in many cases, especially when the caisson can be started on ground, as in Fig. 12-1(*c*). Forms are necessary, but sliding forms, especially when made of steel, can be used efficiently and without unreasonable cost. Quick-setting cement will also reduce the time required before the forms can be stripped and the concrete put under stress. The use of concrete alone for caissons is shown in several of the illustrations. Naturally, as far as materials are concerned, caissons of moderate size built of reinforced concrete alone are often more economical than are those made of structural steel plus concrete fill.

The size, shape, and depth, the construction procedures used, the character and weight of the structure to be supported, the difficulties that may be encountered, available materials, transportation, past experience under reasonably similar conditions, and relative economy—all these are influences that bear upon the choice of the material to be used. The entire program should be thought through carefully before a decision is made.

One of the important details of construction is the cutting edge. This should be simple but strong. Nothing much can be done about it if a cutting edge fails under the local pressure caused by some obstruction 100 ft. or more under water. A triangular or trapezoidal shape is generally desired in order to reduce the bearing resistance of soil trapped under the caisson walls, and to give more chance to remove the ground that is not directly under the shaftways. Great strength is needed to resist compression and twisting from a tendency for the beveled faces to cause the walls to spread out.

Figure 12-7 illustrates some details for the construction of cutting edges for caissons. Some of these are satisfactory; some are of questionable value but are shown purposely. The following comments refer to the various parts of the drawing:

(*a*) This shoe is for a concrete caisson. The outside plate is to provide stiffness and to spread any localized load. The 8 by 8 angle is stiffened by welded plates to hold the downstanding leg straight. The horizontal stiffener angle serves as an aid in spreading loads upward into the concrete, and as an anchor if the cutting edge is forced inward. The continuous tie angle and the top cross ties help to anchor the bracket against forces that tend to push the cutting edge outward. The reinforcement below the cross tie consists of short lapped rods that are threaded through holes in the T's. It seems that the weakest part of the construction is the resistance of the concrete to tension at *A* because the general tendency produced by the beveled face is to force the cutting edge outward.

(*b*) This shoe is strong in itself but very short. Boulders might crumple the plates at *C*, and the anchorage of the steel to the concrete is not very effective. The section is relatively weak just above the brackets. The same scheme made less skimpily would be much better. The offset at *B* is supposed to reduce the skin friction on the face above it. This might be advantageous in highly cohesive soil that is not disturbed; when water or jets practically liquefy it, the offset is of negligible value; and it is not likely to be useful in granular soils. Contact with the soil along the outside faces is usually needed in guiding the caisson. The small seats are supposed to help deliver vertical loads to the concrete seal, but they are relatively insignificant.

(*c*) This illustrates a very simple construction for a cross wall. Since there is access of a sort from both sides, there is not so much need for a heavy shoe. A rounded or sloping concrete section may be sufficient unless

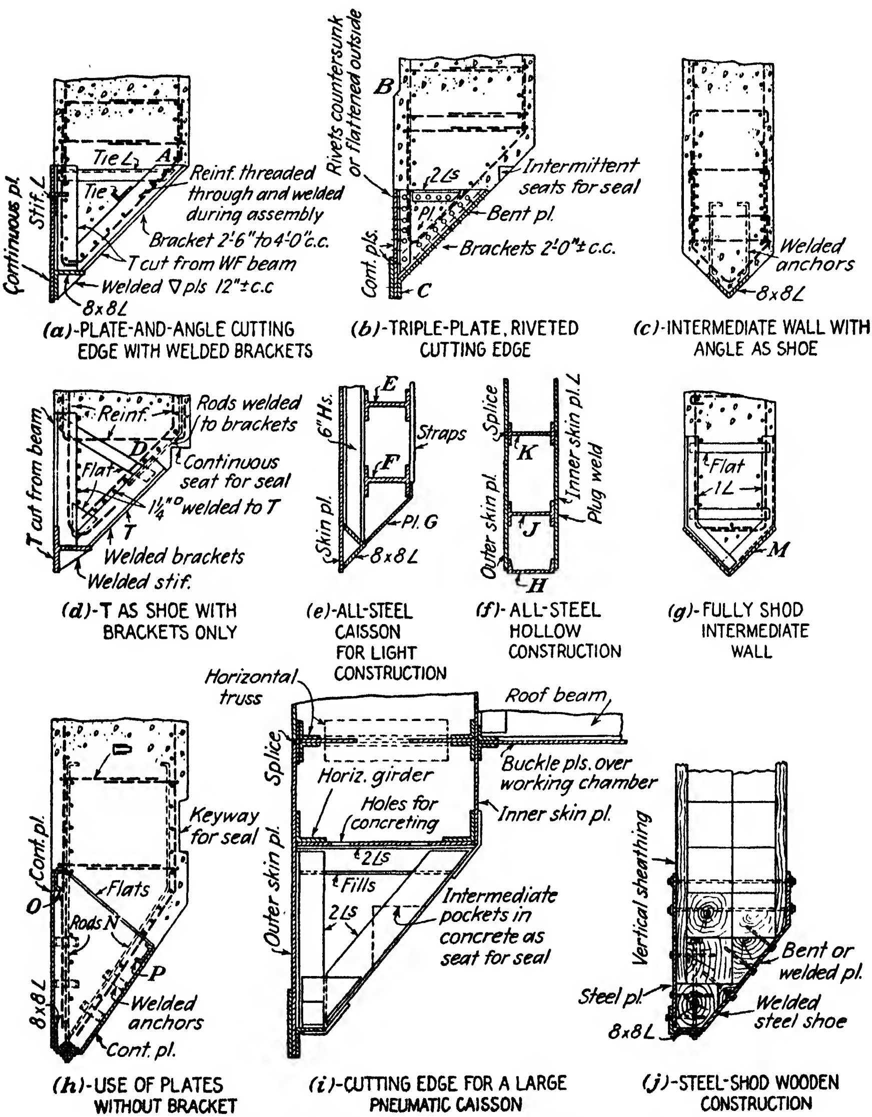


FIG. 12-7. Some details of cutting edges for caissons.

boulders are encountered. The cutting edge of such a cross wall may be placed at a higher elevation than those of the outer walls. This may be helpful when boulders or irregular rock are expected to be encountered.

(*d*) This cutting edge is a heavy T with welded stiffening plates and brackets. The brackets are connected by heavy welded rods that serve as spacers, as anchors against rotation of the cutting edge, and as regular reinforcement. The continuous seat near the upper corner is a better support than that in (*b*), but the corner is likely to cause trouble by catching the excavating equipment. The horizontal web of the T would be a good seat, but the space under it is difficult to fill with concrete when placed by tremie or underwater buckets. The brackets seem to be fairly effective, but they would be better if the T's were extended to reach to and above the inside corner. In the sketch, the welded bent rods are supposed to help remedy this.

(*e*) This shows an all-steel construction for a small caisson. The triangular cutting edge is made by welding an angle to the skin plate. This plate is stiffened by means of vertical *H*'s that cross the wales *E* and *F*. Plate *G* is to hold the cutting edge to wale *F*. Notice the following weaknesses and defects:

1. The skin plate is the only real stiffener for the support of the cutting edge.
2. Sideward pressures against the angle are supposed to be resisted largely by plate *G*, but this merely twists *F* and, through the straps, the other wales. Without vertical diaphragms, the wales cannot offer proper resistance.
3. Plate *G* will almost inevitably bend badly unless it is stiffened by concrete backing.
4. When the concrete seal is placed, the spaces between the lower flanges of *E* and *F* cannot be filled. The webs should be open, as with lacing, but this would weaken them too much.
5. The lightness of the caisson is likely to hamper its sinking. Concrete fill would be useful for this purpose, and for stiffening the construction in general.

(*f*) This sketch shows another type of all-steel construction for small caissons. As shown here, the outer skin plate is not well stiffened, the splices in the skin plates are weak in resistance to large vertical forces. The wales *J* and *K* can be connected to the outer skin plate but, assuming that the inner plate *L* is erected last, the connections of *L* to the wales are weak. Also, the cutting edge *H* is not stiffened satisfactorily. A concrete fill placed by pumping or chuting through holes in the webs of *J* and *K* would improve the construction. On the other hand, a reinforced-concrete caisson might be more economical.

(*g*) This cross wall has a wide shoe of plates *M* with angle anchors. One extra use for the plates may be as a tie between the outer side-wall cutting edges.

(*h*) The cutting edge in this case is not very narrow, but this may be an asset because of its stiffness and resistance to compression. A sharp edge is not really needed unless very firm soils like gravel and hardpan overlie the rock that has to be reached. A certain amount of flatness may be useful in providing a temporary support for the caisson on the ground below it. The plates with their welded anchors and channels are tied into the concrete. The channels *O* and *P* are especially good anchors because they are continuous and are tied together by the flats. Rods *N* are threaded and attached to the cutting edge. This is primarily in order to have them serve as reinforcement that is well anchored at the bottom. On the whole, this construction is simple and sturdy.

(*i*) This sketch illustrates one way of making the cutting edge and working chamber for a large pneumatic caisson built of steel with concrete fill. The steelwork is generally erected in horizontal lifts, riveted and calked for watertightness, or welded. The concrete is then packed in thoroughly in the dry. Since men can get at the ground directly, there is not much likelihood of excessive localized loads under the cutting edge. The entire construction, however, should be very strong. The roof has to act like the bottom of a boat with upward loads that may be 2 or 3 tons per ft.^{2}, and it must be practically airtight. Flooding the inside above the working chamber, or adding "muck," will counteract some of the uplift if necessary, but such steps should generally be reserved as aids in sinking the caisson if unexpected resistances develop.

(*j*) Open caissons may also be made of wood when this is the most available or practicable material. Steel shoes may be used as the cutting edge with heavy timber backing as shown here. The details of the construction should be adapted to the use of commercial timber of standard sizes and lengths in order to avoid needless carpentry work. The details are so special and wood is so often replaced by steel or concrete for the construction of important caissons, that no attempt is made here to show more than this one example.

All the details of caissons should be strong. Design merely to resist external pressures is seldom adequate. Corners should be tied together thoroughly. Cross walls should be integral with exterior ones, or very strongly connected thereto. The walls should be capable of resisting all probable distortions caused by unequal bearing or by anything else that seems to be necessary or probable.

It is important to have the right piping installation for water jets. Experiences with jets in different cases seem to conflict somewhat. It is apparently desirable to have the piping arranged so that the jetting can be localized. For example, referring to Fig. 12-8, the piping for the lubricating jets on the outside is separated from that for the inside. Furthermore, separate installations controlled by valves should be used for each side of a rectangular caisson, or for portions of each side if the caisson is large. In (*a*), the jets on the outside are equipped with nozzles to produce high velocity and to assist in the elimination of clogging, but the inside pipes are large so that portable jetting equipment can be dropped through them. If desired, they can be used to handle a large volume of water and to produce merely a simple washing effect. A series of outer jets is shown in (*b*). The one at the bottom points downward, supposedly to help undermine the region near the cutting edge, whereas the higher ones point upward in the direction that the water will have to take to escape. Portable jets, air lifts, choppers, and other devices may also be handled by derricks at the top. These enable the men to attack a local resistance wherever it may occur. Of course, divers may be employed if the depth permits it.

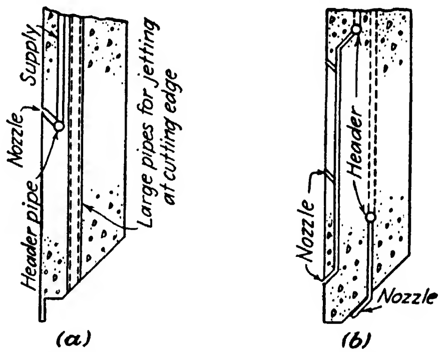


FIG. 12-8. Some arrangements for jetting.

The jets should be spread along the cutting edge at spacings of a few feet. Those to reduce the skin friction outside should be spaced similarly and in rows perhaps 10 to 20 ft. apart vertically.

The big pipes in Fig. 12-8(*a*) can be rodded if they become clogged. If jets get stopped up, there is little that can be done about it. Another difficulty with jetting is that of control. Except right at the nozzles, the water will go in the path of least resistance. It may therefore wash out local channels instead of lubricating the entire surface as intended. It may also disturb the surrounding soil sufficiently to interfere with the steering of the caisson. At the cutting edge, the jets may be entirely ineffective against heavy gravel or boulders that are jammed between them. Compressed air and steam under pressure have been used in lieu of water. In spite of these troubles, jetting is a great help in most cases if it is installed properly and used wisely, but it is not a cure-all.

12-5. Miscellaneous caissons. A caisson may be made as a box with cylindrical or rectangular walls and a bottom. As such, it may be floated to position and then sunk to the natural bottom or to bearing on a previously dredged and prepared area. This sounds easy. However, a level, even, firm bearing surface at the right elevation is very difficult to obtain. Such caissons may be suitable for sea walls or bulkheads, for which they can be sunk and filled with dirt, gravel, rocks, or even concrete. Great accuracy is not usually essential for them, and unequal and considerable settlement may not be too harmful. On the other hand, the dangers accompanying their use as supports of major structures are too obvious to need further discussion.

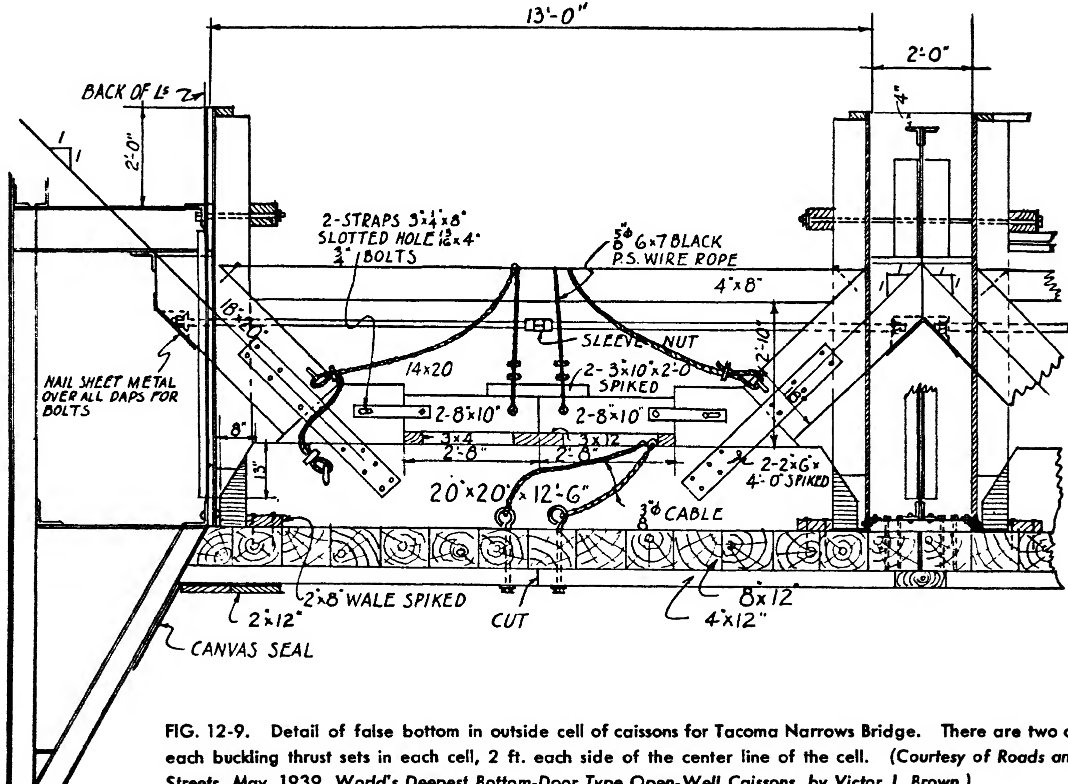


FIG. 12-9. Detail of false bottom in outside cell of caissons for Tacoma Narrows Bridge. There are two of each buckling thrust sets in each cell, 2 ft. each side of the center line of the cell. (Courtesy of *Roads and Streets*, May, 1939, *World's Deepest Bottom-Door Type Open-Well Caissons*, by Victor J. Brown.)

To be able to float a portion of a caisson into place, it may be built with a false bottom that is removed later. The construction of the false bottoms used in the caissons of the Tacoma Narrows Bridge is shown in Fig. 12-9. This is called an open-well bottom-door type of caisson. Each of the two caissons was approximately 66 ft. wide and 119 ft. long in plan. One was eventually about 175 ft. high; the other, 200 ft. The water at one of the pier locations was 120 ft. deep, and tidal currents were as high as 6 to 10 m.p.h.

The construction of the false bottom deserves careful study. After the caisson was floated to position, anchored, and sunk to the prepared level patch in the river bottom, the cells were flooded to eliminate buoyancy. Then a hook was lowered into them individually to catch hold of the wire ropes. When the hoist yanked the ropes upward, the sloping kick blocks were removed and the whole bottom of a cell was "disintegrated" and pulled up. The excavation then proceeded by the use of clamshell buckets working through the water.

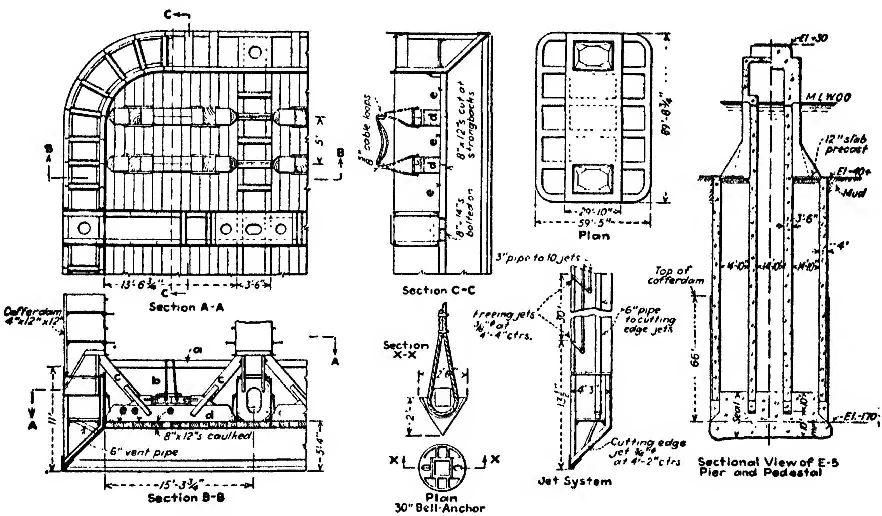


FIG. 12-10. Details of caisson and false bottoms in dredging wells, and general design of pier. *(See Purcell, Andrew and Woodruff, Deep Open Caissons for Bay Bridge, Engineering News-Record, Aug. 23, 1934.)*

Figure 12-10 illustrates somewhat similar false bottoms used successfully for part of the foundations for the San Francisco–Oakland Bridge. It also shows the general features of the caisson and pier construction.

It is obvious that the planning of such false bottoms must be done with great care and thoroughness. It is vital that they operate as intended, that they be strong enough, and that mishaps be avoided. Apparently the tipping of the caisson shown in Fig. 12-28 was the in-direct result of trouble that developed when trying to remove the false bottom.

A very interesting type of caisson for work in deep water is shown in Figs. 12-11 to 12-13. It was invented by Moran and Proctor and was used for the San Francisco–Oakland Bridge. It shows a means of controlling a caisson through the use of "false tops" instead of false bottoms. It consists of a structural-steel open caisson with cylindrical wells to which steel domes can be welded. Compressed air forced into the capped wells tends to buoy up the caisson. In this way a large portion of it can be fabricated and floated to position. Partial release of the air will permit the caisson

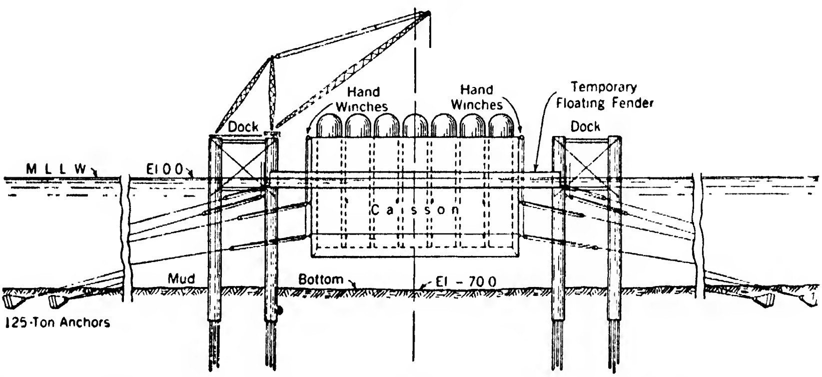


FIG. 12-11. General picture of caisson used for constructing pier of San Francisco–Oakland Bay Bridge, showing caisson, working docks, and temporary floating fender system. *(See Carlton S. Proctor, Constructing Foundations of Trans-Bay Bridge, Civil Engineering, Feb., 1935.)*

to be sunk until it reaches the bottom. Having many cells, it is possible to build up the steelwork in lifts with compressed air in part of the wells to provide buoyancy. The open ones can then be extended at the top, capped, and filled with air. The caps can then be removed from some of the remaining wells, the tops extended, and the domes welded on again. This is repeated until a complete lift has been added. Thus the caisson can be water-borne, even when being sunk, until the bottom is reached, and even later if necessary.

Thereafter, excavation can be made through some of the wells while the others remain under air pressure. Capping of the open ones, removal of the domes from others, and excavation through the newly opened wells permit more of the ground to be removed from under the caisson. By continuing such work in series until the desired excavation has been made under the caisson, permanent tipping and loss of support can be avoided or largely corrected. When it is ready to sink to another level, the air pressure can be reduced and the caisson's sinking controlled. Such work can be repeated until buoyancy is no longer needed. Then the excavation and sinking can be carried on as for an open caisson.

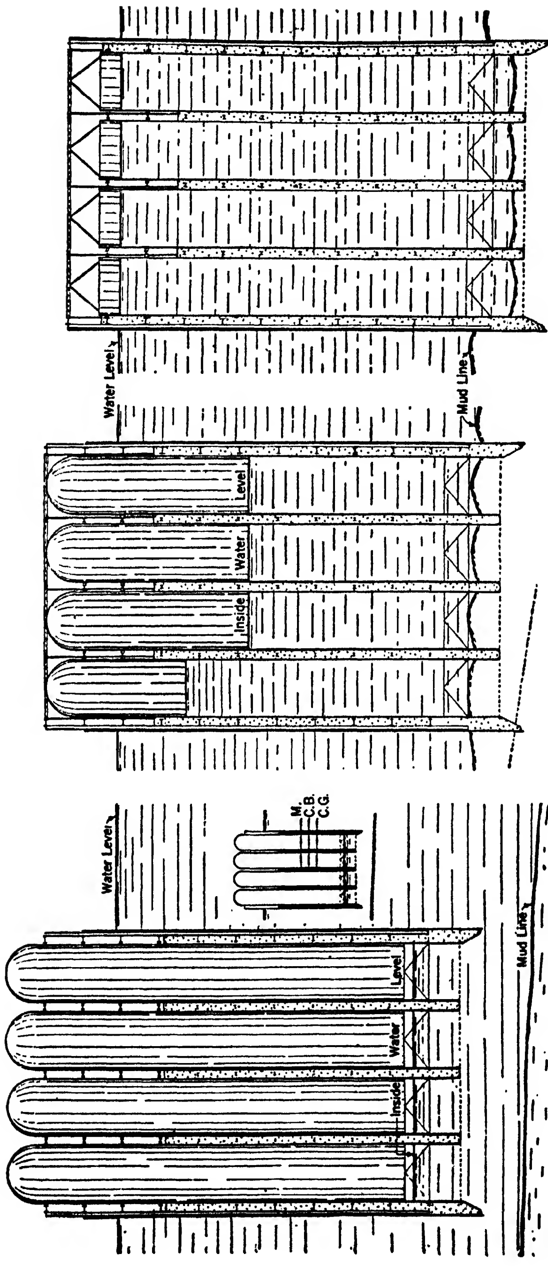


FIG. 12-12. Floating, landing, and excavating within domed cofferdam for San Francisco—Oakland Bay Bridge. (For this and Fig. 12-13 see Carlton S. Proctor, *Foundation Design for Trans.-Bay Bridge, Civil Engineering, Dec.,* 1934.)

A substructure may be built by means of a caisson and a cofferdam combined. One such case for the construction of a bridge pier is illustrated in Fig. 12-14. The cofferdam fitted on top of the caisson as shown. It was removed later. This combination may be economical for caissons of large

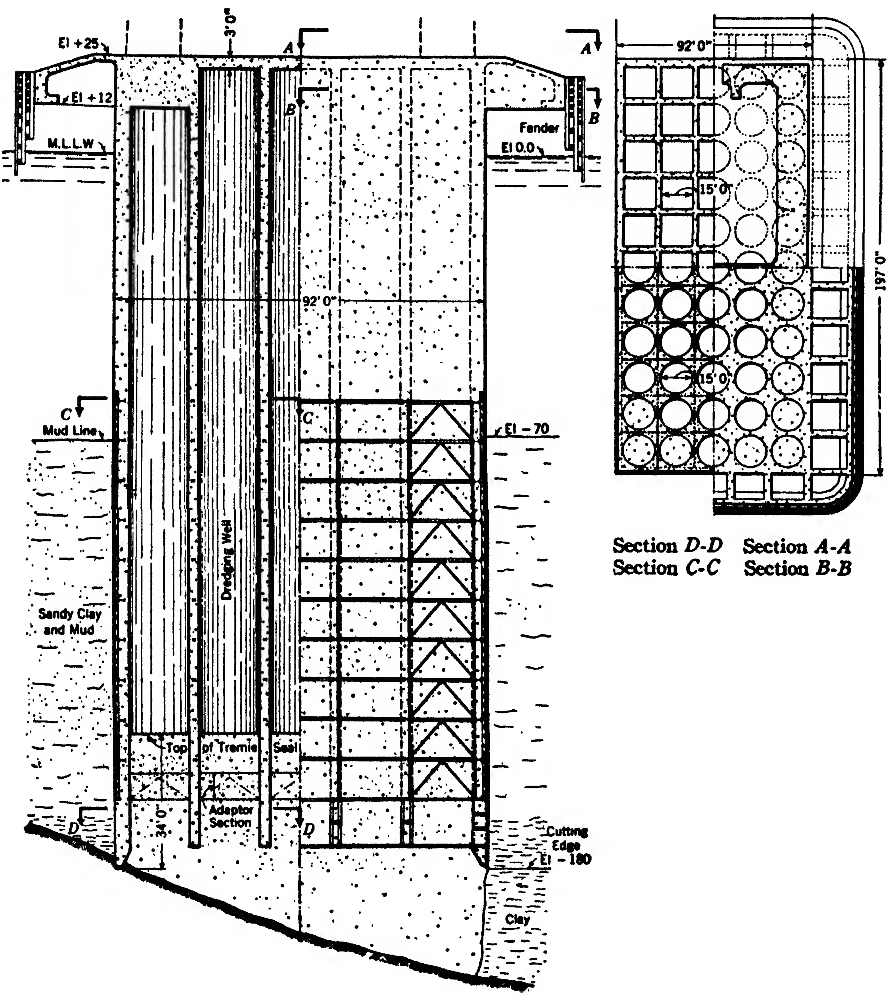


FIG. 12-13. Landing and sealing of caisson for San Francisco-Oakland Bay Bridge.

area when conditions permit it. There is sometimes a real advantage in keeping the projecting caisson out of sight, provided it is below the draft of vessels which might otherwise be wrecked upon it.

Figure 12-15 pictures a caisson of timber construction with a steel cutting edge, concrete walls and partitions, false bottoms, and a cofferdam top. The structure had to be supported upon very deep river sand. The maximum tidal and river current was approximately 13 f.p.s. The caissons were towed into three-sided pile enclosures to hold and protect them. Notice the use of gravel in the lower portions of the cells, even beneath the concrete that is in the cell directly below the pier shaft. The gravel and stones surrounding the caisson were added to prevent scour.

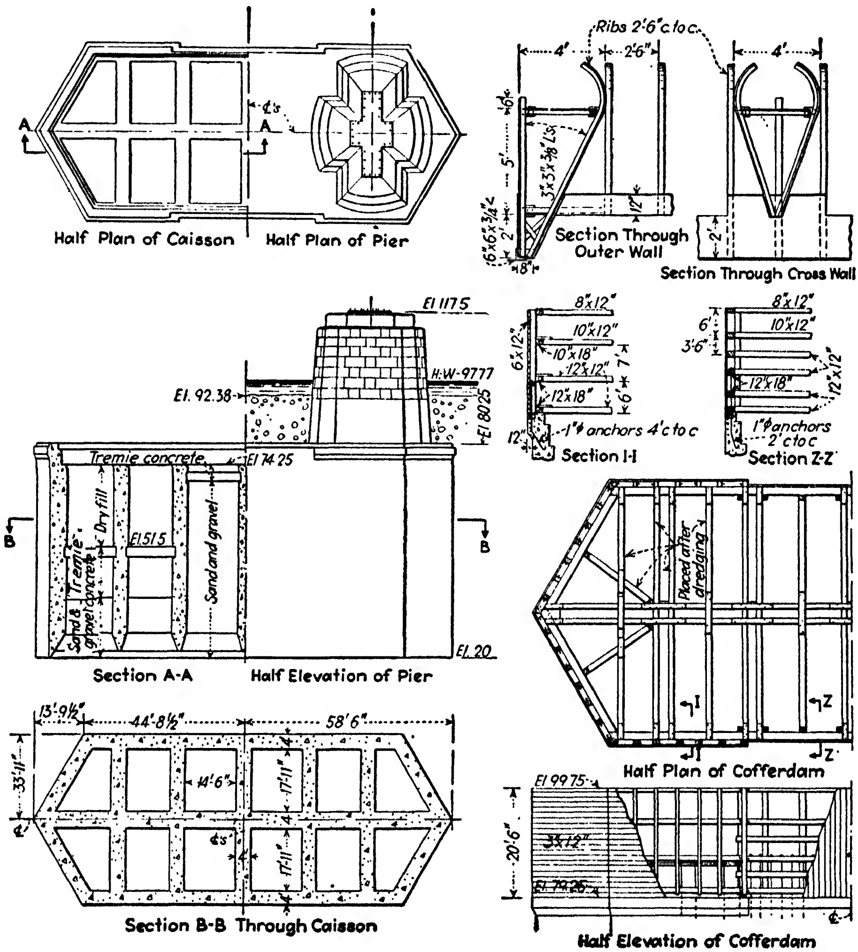


FIG. 12-14. Details of north pier caisson and cofferdam for the Lion's Gate Bridge at Vancouver. ***(See Convertible Caissons for Bridge Piers, Engineering News-Record, Feb. 16, 1939.)***

Caissons need not be large. A type that is very useful for the foundations of large buildings consists of a steel pipe from 2 to 6 ft. in diameter that is sunk into the ground. The earth inside is removed by ordinary methods of excavation under water, by washing with water jets, or by blowing out the contents by the explosive action of compressed air. In some cases, the rock at the bottom can be cleaned by divers but, when the depth is too great, washing and clamshell buckets may have to be depended upon. It is obvious that boulders may cause serious difficulties in sinking such small caissons. After cleaning, the caissons are filled with concrete. When the soil is sufficiently cohesive, the steel shells may be withdrawn as the concrete is deposited.

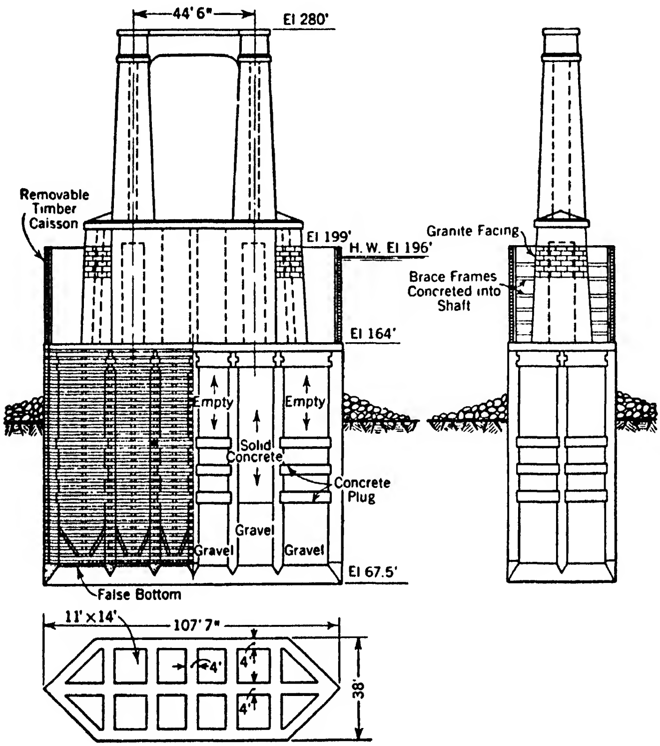


FIG. 12-15. Pier and caisson construction at Pier No.3, Fraser River Bridge. (*See Some Recent Canadian Construction, Civil Engineering, Dec.,* 1937.)

A variation of these small cylindrical caissons is shown in Fig. 12-16(*a*). The steel pipe is equipped with hardened teeth at the bottom. Special equipment is used to rotate it as it sinks, thus cutting its way through obstructions and firm soil to rock. On the other hand, boulders and coarse gravel may interfere seriously with the sinking process because of their tendency to act like ball bearings.

The small caisson shown in Fig. 12-16(*b*) pictures a type that was used for a pier at Seattle, Wash. The overburden on top of good gravel was too thin to support piles laterally, and the load was so heavy that fewer but stronger supports were desirable. The overburden was first removed by dredging the area. Then the working chamber of concrete was precast, the removable steel form for the main column was attached thereto and calked, and the reinforcing cage was inserted. The assembly was lowered to the bottom by a floating derrick, plumbed, and braced temporarily. Weight for sinking was provided by an 80-ton water-ballast tank surrounding the air lock that was bolted to the top of the form. Compressed air

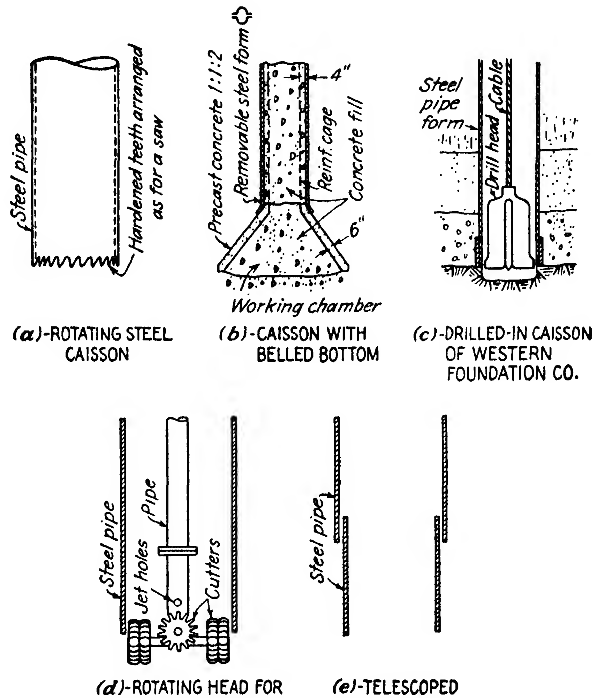


FIG. 12-16. Some details and construction methods for small cylindrical caissons.

forced the water out of the shell. The sandhogs then excavated down to firm material and prepared for the concreting. All concrete was placed in the dry and compacted by the sandhogs. The concrete in the bell was allowed to set before the main shaft was poured. The forms were stripped later, and the cylinders were inspected by divers.

Figure 12-16(*c*) shows a way of preparing the bottom of a pipe caisson. The well-drilling type of head is used under water to grind or chop to pieces any boulders or poor rock. After cleaning, the concrete fill is placed directly upon the sound rock. The pipe may or may not be removed.

Another type of drilling head is shown in Fig. 12-16(*d*). It was used to sink the shafts for the Montgomery Ward & Co. building in Chicago. They penetrated 70 to 80 ft. of clay and boulders. The cutters ground out the clay so that the jets of water emitted from the hollow shaft could wash it away.

A telescoped arrangement of the pipe casings, as shown in Fig. 12-16(*e*), may be helpful in reducing the skin friction to be overcome when sinking pipe caissons. The upper section may be driven as far as it is practicable to do so, then a slightly smaller shell may be driven inside of the first, by means of a follower. If leakage of ground between the two pipes is feared, the clearance may be reduced, or some type of closure ring and calking may be used at the bottom of the higher section. In the Gow system, the bottoms of these caissons are belled out to form a pedestal and increase the bearing area.

The known presence of boulders, the suspected existence of buried obstructions, the likelihood of sloping or irregular rock surfaces, and the necessity for the removal of considerable broken or disintegrated rock in order to reach good bearing—these are things that make it desirable to plan a caisson so that, if it is found to be impossible to sink it by means of open dredging, it can be converted easily to use as a pneumatic caisson.

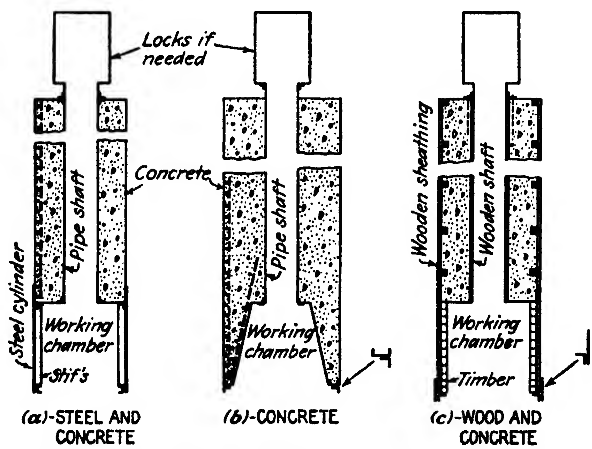


FIG. 12-17. Some small open caissons for heavy building work planned for pneumatic work if found to be necessary.

Figure 12-17 pictures a few types of construction that were planned with this in mind. They are small caissons that are to support heavy building columns. In (*a*) the working chamber and cutting edge combined is a strong steel cylinder underneath the cylindrical reinforced-concrete portion of the main caisson. The shaft is to be used for open excavation, or it may have the necessary locks attached to its top if compressed air is needed. Sketch (*b*) pictures a round or rectangular caisson of reinforced concrete except for the steel shoe and pipe shaft. The construction in (*c*) is a square or rectangular caisson with a steel-shod wooden structure as a bottom unit that is convertible into a working chamber. This may be suitable when in clay where leakage of air may not be serious. In other soils, it may be difficult to keep from flooding the working chamber. It is also obvious that the wooden frame will not have great resistance to crushing if the caisson becomes hung up on a boulder.

When planning the spaces that may become future working chambers, one should be careful to give a man space enough in which to work. A height of 8 ft. is a desirable minimum. A width of 4 ft. is about the narrowest for a man handling a shovel. Even this provides very inadequate space when he is supposed to shovel dirt and lift rocks into a bucket under the shaft.

At the South Meadow Station of the Hartford (Conn.) Electric Light Co., cylinders 5 to 6½ ft. in diameter were to be sunk approximately 110 ft. to rock. Above the rock there was about 10 ft. of hardpan that had water under considerable pressure, 65 ft. of very soft clay, 25 ft. of sand and gravel, and miscellaneous fill and sand. The Powell method was used. Briefly, the procedure was the following:

1. A hole slightly larger than the caisson was drilled down to the hardpan by means of a three-pronged rotating bit equipped with five bolted teeth on each arm.

2. The hole was kept filled with heavy mud to prevent caving-in of the surrounding soil.

3. When hardpan was reached, the bit was withdrawn, and a steel pipe or shell was dropped down through the mud. The bottom of this shell was a double-wall core barrel.

4. The shell was rotated, and chilled steel shot were fed into the cutting edge. This continued until the cylinder was supported on sound rock. Most of the waste material was washed out.

5. The hole was pumped dry, cleaned to rock, and prepared for concreting. This was done while entering water was being pumped out continuously.

6. The hole was then filled with water, a seal of tremie concrete was placed, the rest of the water was pumped out, and the shaft was filled with concrete. Provisions had been made for compressed air, but its use proved to be unnecessary.

These small caissons may not be heavy enough to be sunk under their own weight. Figure 12-18 illustrates one case where jacks were used. Sometimes heavy weights are applied, somewhat as for a tremendous loading test for soil. More often the casings are driven somewhat as piles would be. Cast-steel driving hoods, cushioned frames of steel, heavy billets 6 in. or more thick, and special extensions are used to protect the edges of the pipes from being battered. Ring stiffeners may be necessary inside to stiffen the pipes. When sealing to the rock is difficult in open air, the pipes make it easy to convert them to pneumatic work by adding air locks at the top.

12-6. Special construction procedures and problems. When a concrete caisson is to be started on top of soft ground, it is generally necessary to hold it up in some manner until it has been built up to a height of perhaps 10 or 15 ft. One way to do this is to set the steel shoe on a series of closely spaced transverse timbers that will serve somewhat as a footing when the first few lifts of concrete are poured. These timbers should have blocking added to support the heel of the beveled edge of the caisson as soon as it is built, thus spreading the load and relieving some of the pressure on the cutting edge. A seat like that in Fig. 12-7(*d*) is useful for bearing on this blocking. When the caisson is ready to be sunk, the blocking timbers should be removed individually in such a manner as to maintain reasonable equality of pressure per linear foot of the cutting edge and to prevent tipping of the caisson. Heavy tackle may be needed to pull out the grillage timbers, especially the last few that will be scattered under the cutting edge and probably squeezed into the ground.

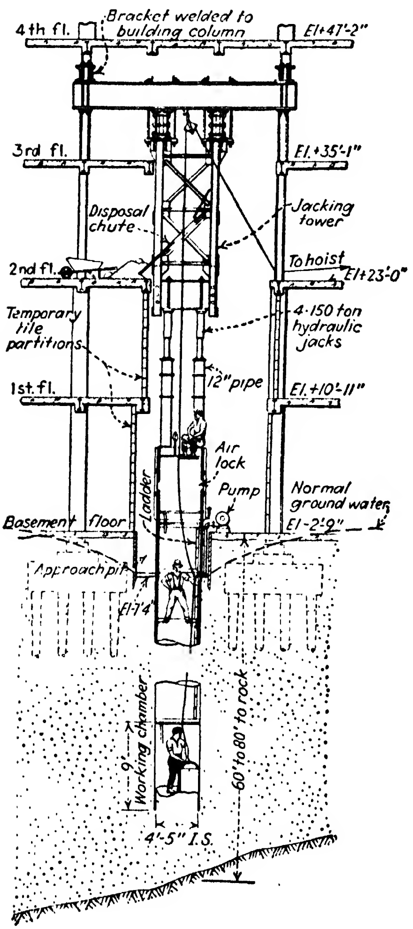


FIG. 12-18. Sinking pneumatic caissons by means of four 150-ton jacks reacting against the weight of a building through a special jacking frame for a viaduct built through the Bell Telephone Laboratories in New York. *(See George T. Gilman, Railroad Viaduct Carried through Occupied Building, Engineering News-Record, Nov. 30, 1933.)*

Jacks and shims, or wedge jacks, are very useful in assembling the cutting edge. They enable the operators to make sure that the cutting edge is level.

The sand island is a very helpful device during the sinking of a caisson, especially where shallow water, tides, floods, waves, current, or deep muddy bottom makes the work hazardous or difficult otherwise. One such arrangement is pictured in Fig. 12-19. In this case, part of the gravel of the river bottom was excavated as shown, and the space was backfilled with pea gravel. This was an effort to reduce the skin friction on the caisson. An island was built above high water so that the caisson could be started in

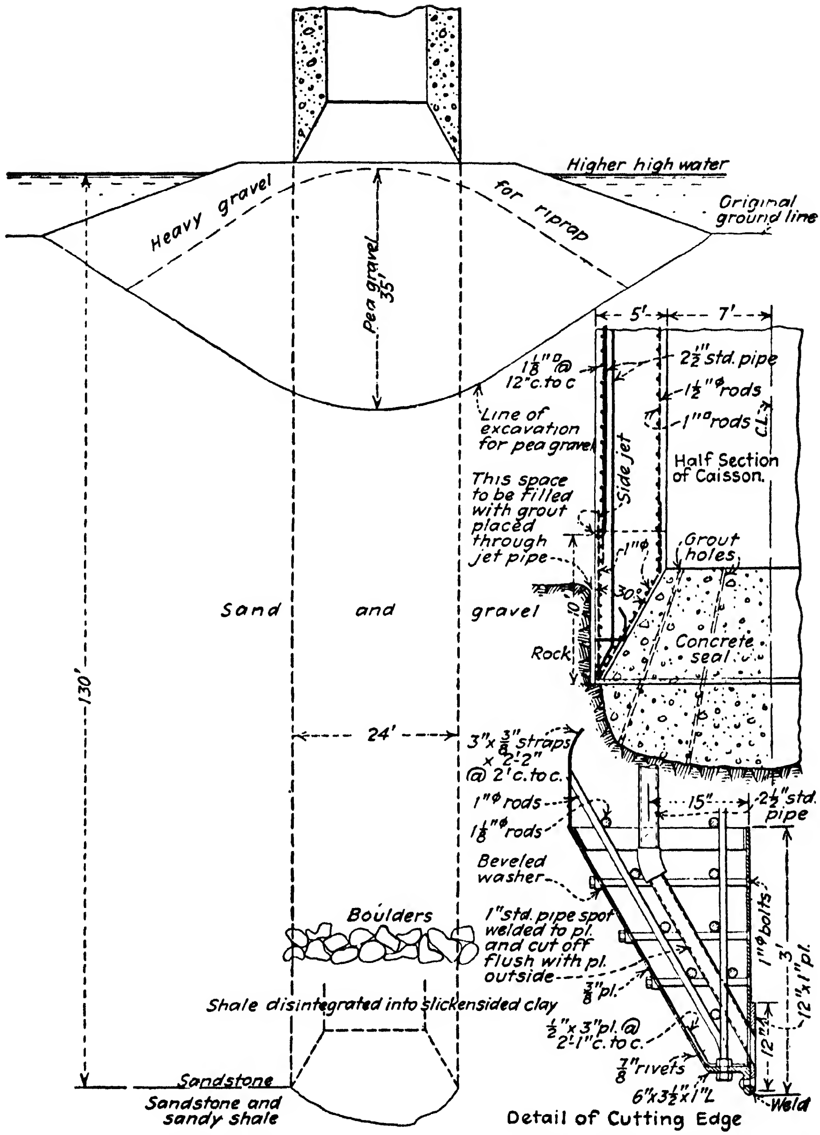


FIG. 12-19. Materials encountered at and details of construction used in a caisson sunk to a water tunnel at the entrance to the harbor at Vancouver, B.C. (See W. Smaill and R. Wynne-Edwards, *Difficult Caisson Sinking for Vancouver Water Tunnel, Engineering News-Record, July 6, 1933.*)

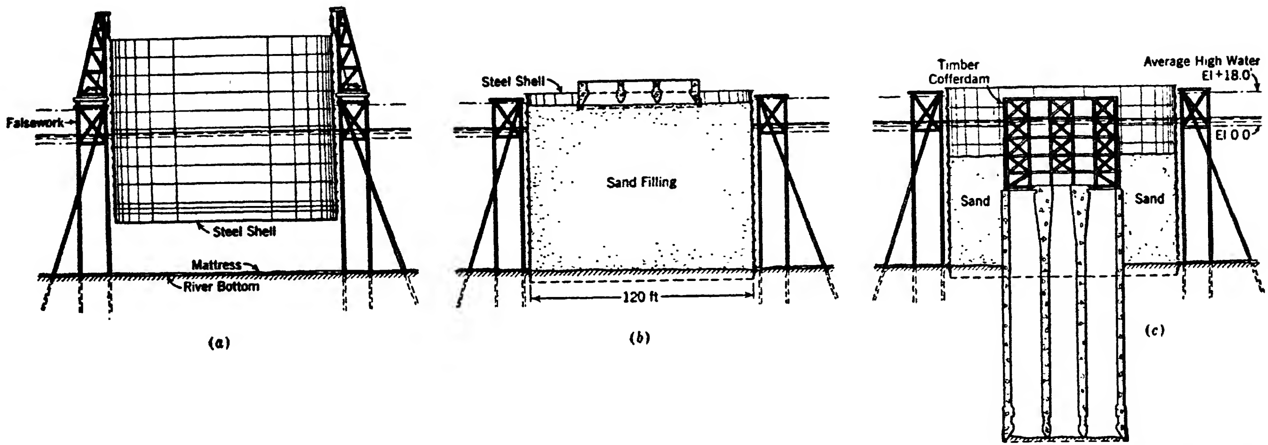


FIG. 12-20. Three steps in the sinking of the caissons of the Huey Long Bridge at New Orleans. (a) Steel shell for sand island being lowered; (b) Shell completely filled with sand and first lift of caisson in place; (c) Timber cofferdam constructed and dredging operctions nearly completed. (See *Foundations for the New Orleans Bridge*, N.F. Helmers, *Civil Engineering*, July, 1936.)

the dry, and a good working space would be available for future operations. The caisson was thus embedded and under control at all times.

A more spectacular sand island is pictured in Fig. 12-20. The shell for such an island may consist of a circular ring of steel sheet piling or steel plates with a sand fill placed inside it and resting on the river bottom. This makes a temporary island with its top above water. The caisson can be erected on this ground which constitutes an excellent working platform. Then it can be sunk down through the fill and the soil below. The great mass of the island prevents disturbance. Once the caisson is well started and "aimed" correctly, it can be controlled more easily for the remainder of its course. After the caisson is in its final position and the bridge pier is finished, the piling and sand can be removed.

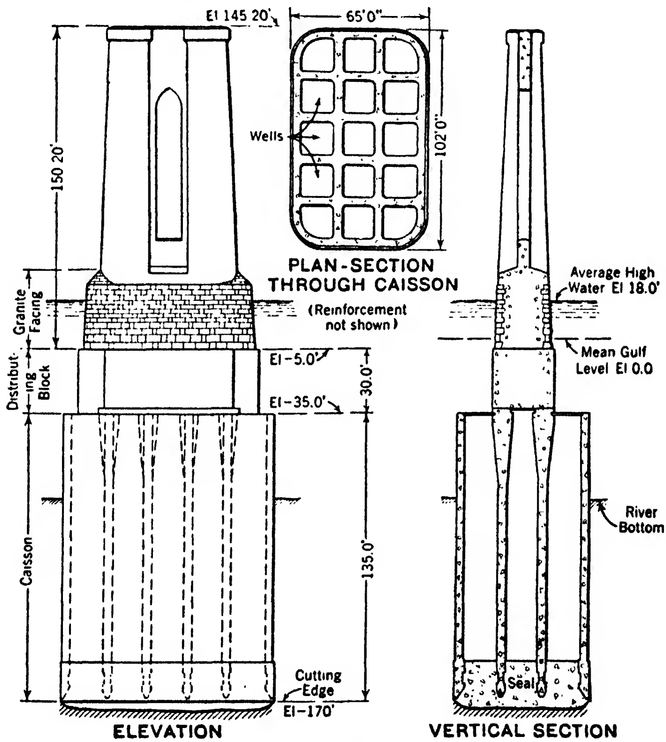


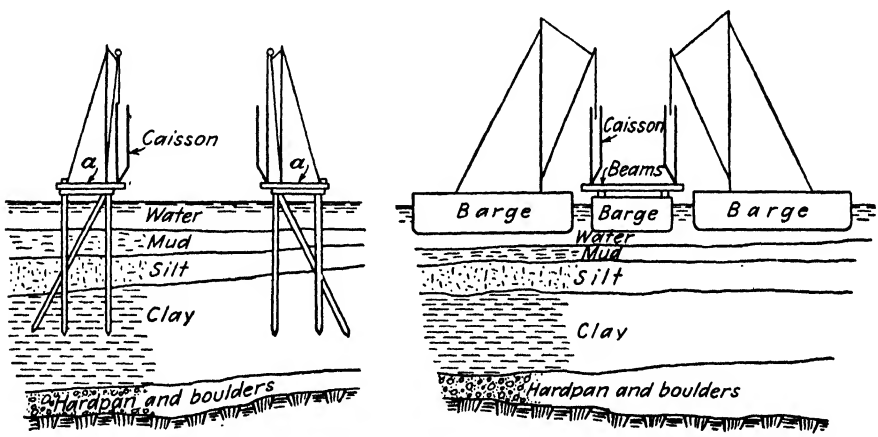
FIG. 12-21. One of the piers of the Huey Long Bridge at New Orleans, La. (See *Foundations for the New Orleans Bridge*, N.F. Helmers, *Civil Engineering*, July, 1936.)

The sand islands shown in Fig. 12-20 were used in the construction of the main piers of the Huey Long Bridge across the Mississippi River at New Orleans, La. The bridge is in a region where severe floods may occur, with flood stages of 15 ft. or more. At one of the piers the water was 75 ft.

deep. The river bottom is composed of layers of sand and mixtures of sand and clay. Firm sand was at least 170 ft. below river level. There was serious possibility of overturning of an ordinary open caisson. In this case, willow mattresses were sunk to the river bottom and weighted with riprap to prevent scour. A steel shell 120 ft. in diameter was built in 13-ft. lifts inside a ring of falsework, then it was filled with sand. In this case, an open cellular caisson of steel and concrete was used. In order to reduce the weight of the pier because the caisson was supported upon earth, the caisson was left hollow, as shown in Fig. 12-20 for the caisson alone and in Fig. 12-21 which pictures one of the completed piers.

When this bridge was erected, the total settlement was approximately 4 in. It is interesting to note that the piers sink somewhat when the river is at flood stage but they rise again when the flood subsides. Perhaps this is caused by compaction of strata of soil below the bases because of the surcharge effect of high water. However, the real causes remain mysterious.

Guiding an open caisson during the early stages of its sinking may be difficult if it is erected in fairly deep water or in shallow water over deep mud or silt. Figure 12-22(*a*) illustrates the use of a pile platform around a



(*a*)-WORKING PLATFORM ON PILES (*b*)-WORK CONDUCTED FROM BARGES

FIG. 12-22. Two schemes for conduct of work of starting light caissons.

light steel or wooden caisson. Temporary beams *a* are anchored to the platform and cantilevered to support the cutting edge and the first few lifts of the shell. When enough of the caisson is erected, it is lifted, the beams are removed, the shell is lowered to rest on the bottom, more lifts are added, and some of the concrete fill may be placed. Guides are then connected to the platform. They, through the strength of the batter piles, are supposed to hold the upper part of the caisson in line until it is deeply embedded so that the soil will steady it. It is obvious that excavation, and especially jetting, may disturb the support for the inside row of piles. Furthermore, the piling cannot resist the force caused by any serious tipping.

Another case is shown in Fig. 12-22(*b*). The water here is deep enough to permit the use of floating equipment. The lower portion of the caisson, if not too large and heavy, may be erected directly upon an anchored barge, or this may be done at some dock and the barge may be towed to the site. Derrick barges may then be anchored alongside. The derricks can

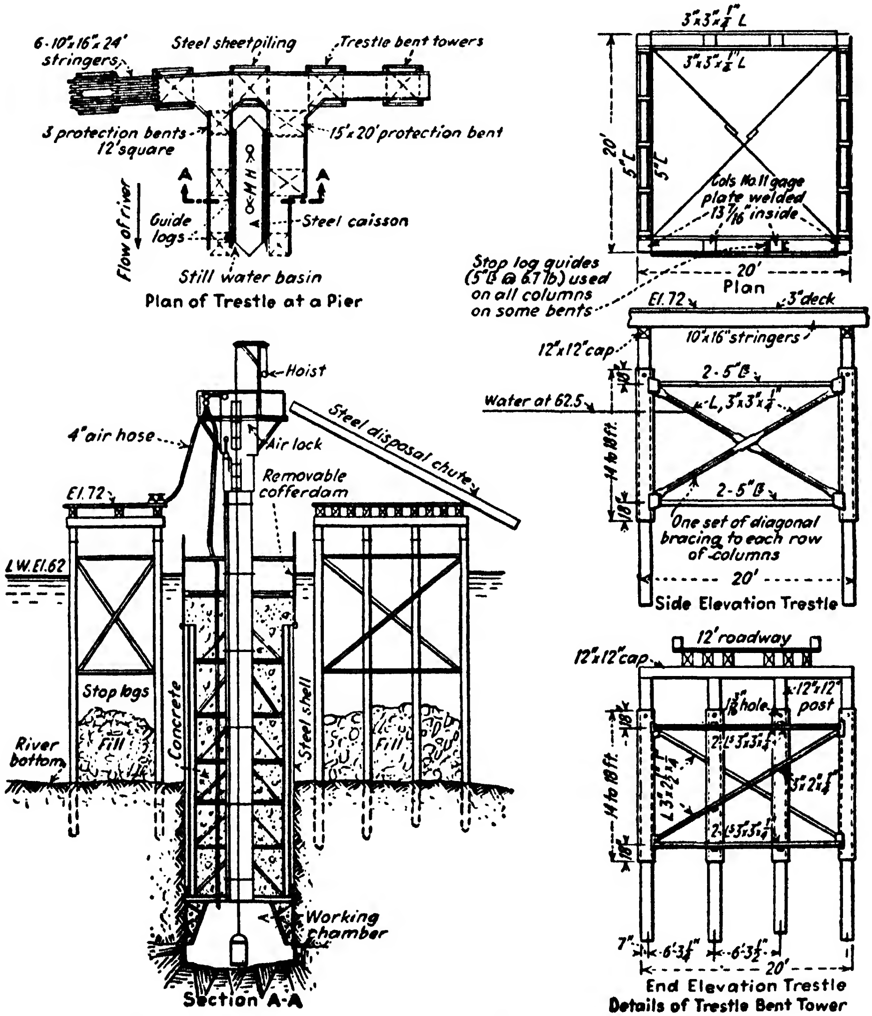


FIG. 12-23. Details of prefabricated steel tower bent trestle construction used on Caughnawaga Bridge. *(See George L. Freeman, River Piers Built with Land Plant from Trestle of Unique Design, Engineering News-Record, May 17, 1934.)*

lift the caisson shell so that the supporting barge can be pulled out, and they can then lower the shell until it rests upon the mud and silt. It is obvious that the site for such work should be well protected from large waves and that good anchors are needed.

Another way to handle the field work is shown in Fig. 12-23. A series of prefabricated steel tower bents with hollow posts were made so that timber spuds could be driven through them, and the steelwork could be supported

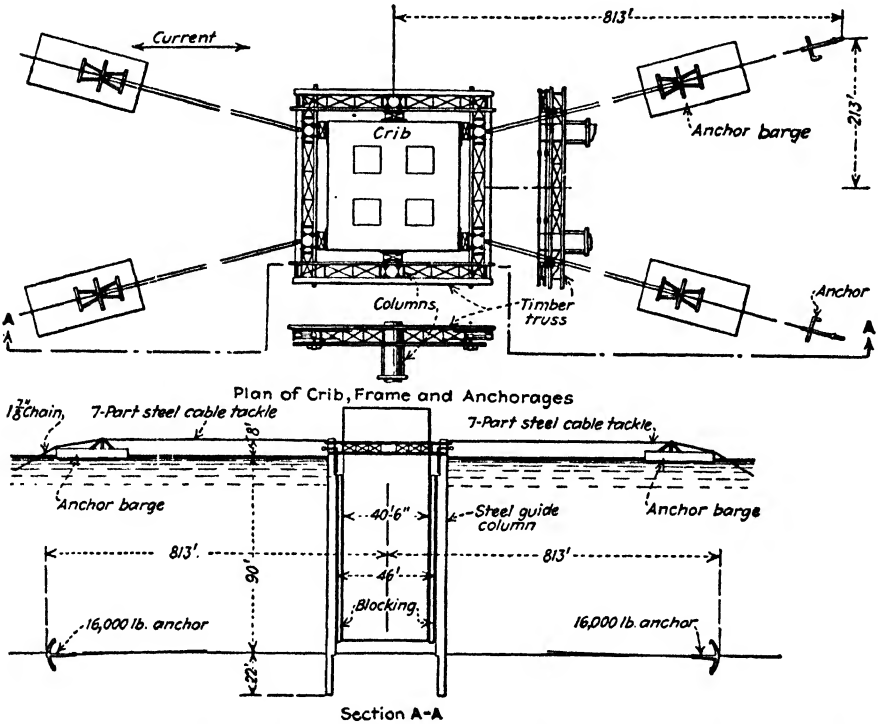


FIG. 12-24. Anchorages and guides for sinking caissons of Carquinez Strait Bridge. (See George J. Calder, Deep Water Caisson Sinking, Carquinez Strait Bridge, Engineering News-Record, Mar. 24, 1927.)

by these piles. A trestle was thus built out to the site of the pier, and a working platform was constructed as indicated. Land equipment could be used. This was a great advantage because of the current, storms, and floating ice. Fenders and cribs protected each caisson against the current. The working chambers of these pneumatic caissons were assembled on timbers spanning transversely across the sites for the piers. When built to a height of 14 ft., they were lifted, the timbers removed, and the caissons lowered into the water.

In cases like that of Fig. 12-24, it is necessary to depend upon ships' anchors or some other kind of anchors to hold a caisson in position. Six

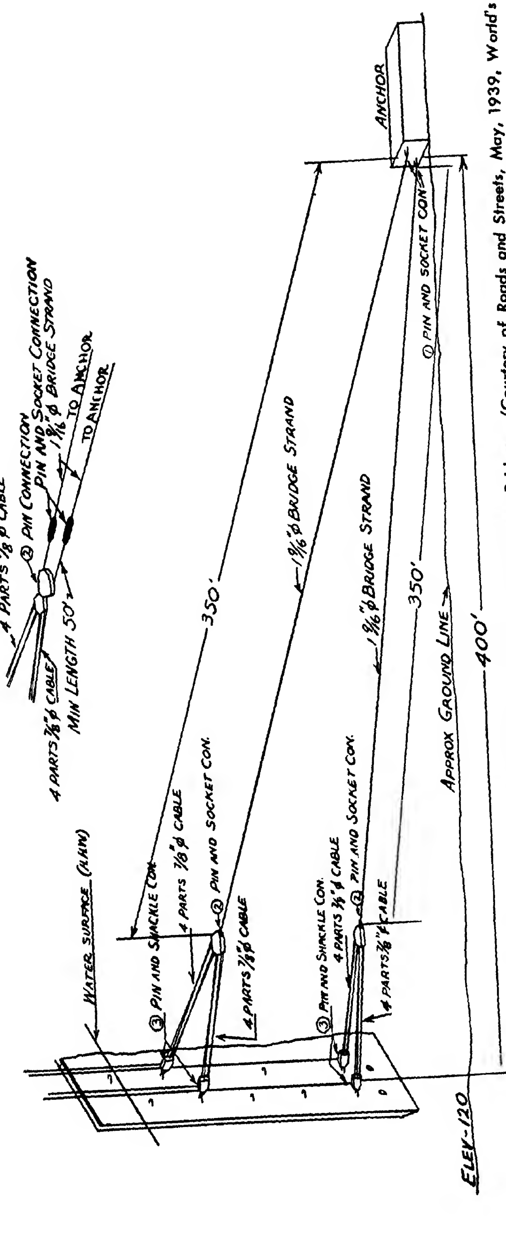


FIG. 12-25. Details of attachment of anchors to the caissons for the Tacoma Narrows Bridge. (Courtesy of Roads and Streets, May, 1939, World's Deepest Bottom-Door Type Open-Well Caissons, by Victor J. Brown.)

30-ton structural-steel guide columns 120 ft. long were driven as shown. After five of these were in place, the caisson was floated into position. The last column was then driven. Steel girders, tie rods, and timber trusses were attached to the columns to form an enclosing frame. Four 1-in. steel cables were attached to yokes on the respective columns and to heavy anchored chains on barges to keep their ends out of water. The sheaves permitted the lines to be tightened as necessary. The caisson was then sunk within this framework.

The caissons for the Tacoma Narrows Bridge presented a serious anchorage problem. The great depth of water, the current of 6 to 10 m.p.h., and the 17-ft. tide range made a tough combination for such large caissons.

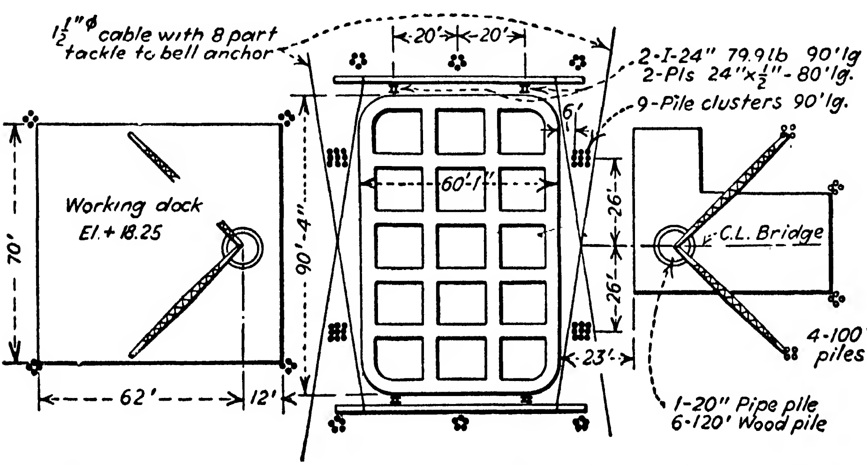


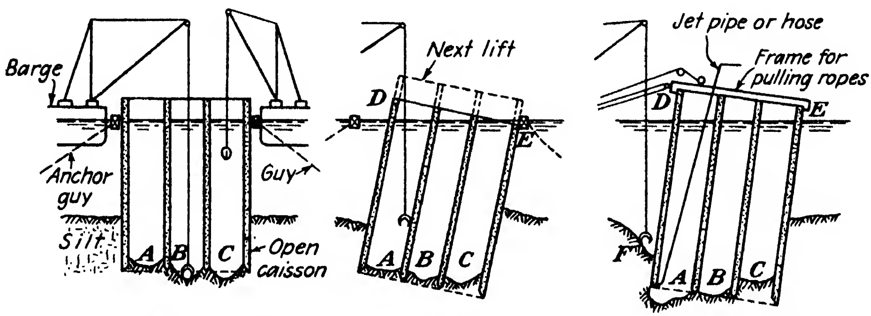
FIG. 12-26. Working docks and system of anchor lines to hold a caisson during the early stages of sinking. *(See Purcell, Andrew, and Woodruff, Deep Open Caissons for Bay Bridge, Engineering News-Record, Aug. 23, 1934.)*

The anchors were large blocks of concrete, weighing 600 tons each. Twenty-four of them were sunk so as to form approximately a 900-ft. circle around the site. Cables were attached to them and to the caisson at various elevations in order to prevent tipping. The connections and arrangement of a typical anchor are shown in Fig. 12-25. The cables could be tightened to adjust for the change in length as the caisson moved downward. The anchors were cast on barges. When the chambers on one side of the barges were filled by opening the sea cocks, the barges were tilted enough to let the anchors skid into the water. They landed within 12 ft. of their planned positions.

Measurements made to estimate the tension in the cables seemed to show that the pulls were only approximately 50 per cent of the maximum computed values used in the design. This may be in part the result of the water tending to form its own streamlining by trapping some of it in front of the upstream face of so large a surface. Nevertheless, the plans should always be made to ensure safety, as these were.

Figure 12-26 shows another arrangement for the anchors and working platforms for a caisson of the San Francisco–Oakland Bridge.

Once a caisson gets out of line, correcting the tilt is likely to be a very serious problem. To illustrate some of the troubles, refer to Fig. 12-27. Sketch (*a*) is greatly simplified to show one case schematically. Supposedly, the excavation in cells *B* and *C* was considerably ahead of *A*. This lack of equal support then caused the caisson to tilt to the right as pictured in (*b*). The top edge *D* moved away from its guide, whereas *E* pushed its guide along, slackened the guy, and became submerged.



(*a*)-CONDUCTING EXCAVATION THROUGH CELLS (*b*)-CAISSON IN TILTED POSITION (*c*)-CAISSON BEING RIGHTED

FIG. 12-27. Illustration of difficulties of righting an open caisson that has tilted.

The first remedial step probably would be the addition of the next lift in order to prevent submergence, although this may not be strictly necessary. Sketch (*b*) shows the difficulty in trying to excavate cell *A* by clamshell buckets in order to undermine that edge. The bucket will reach only the space next to the partition. Some possible procedures are shown in (*c*). The soil at *A* may be loosened by jetting under that edge. The ground outside at *F* is removed in order to reduce the passive pressure against counterclockwise rotation back to the vertical. Cables attached to anchors or to shore may be able to pull the top over as the undermining proceeds.

An unusually tough problem of correcting a tilted caisson is illustrated in Fig. 12-28. Uplift by a barge, weighted booms, pull on cables, hung weights, and undermining by an ingenious bucket system eventually succeeded in righting the caisson so that it was finally only 12 ft. away from its intended position. This illustrates foundation problems that fully tax the ingenuity of engineers.

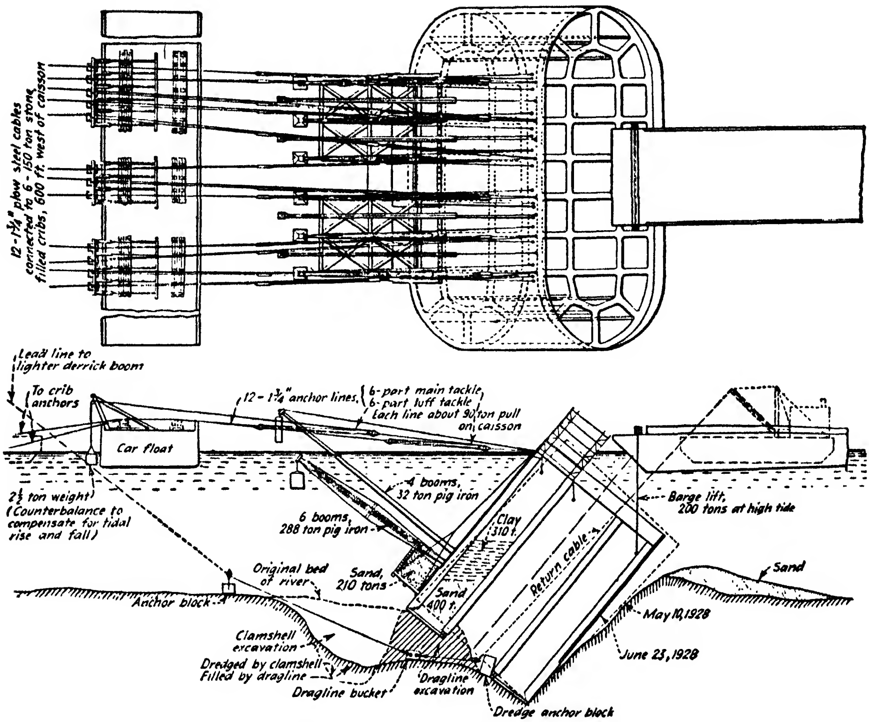


FIG. 12-28. Methods used in righting a large caisson. Weighted booms and pulling tackles anchored to sunken cribs were rigged to apply a pull of 1,500 tons at top of caisson. *(See Glenn B. Woodruff, An Overturned 19,000-ton Caisson Successfully Salvaged, Engineering News-Record, Feb. 12, 1931.)*

PROBLEMS

The following problems are for practice in the planning of construction procedures for the building of deep foundations. It may be that some type of caisson is desirable in all these problems, but not necessarily so. If a caisson is to be used, what type, what general dimensions, and what procedures are the most desirable? All bridge piers have considerable transverse and longitudinal overturning moments at their bases because of wind forces. This should be borne in mind when planning the foundations.

12-1. Figure 12-29 shows the soil profile at the site of a tall bridge pier. The section is taken parallel to the bridge axis. The pier load at the base of the shaft is approximately 3,200 kips. Tidal current is slight. Wave action is not important. Plan the foundation.

12-2. It has already been determined that a large caisson is to be investigated for the construction of the pier to be built on the rock shown in Fig. 12-30. Tides and tidal currents are large, but the waves are moderate in size. The bridge is very heavy. Plan the construction, and explain the reasons for it.

12-3. Figure 12-31 shows a longitudinal profile at the site of a bridge pier across a protected portion of a meandering river near tidewater. The pier is to have two shafts, each 8 ft. longitudinally and 6 ft. transversely of the bridge at mean high

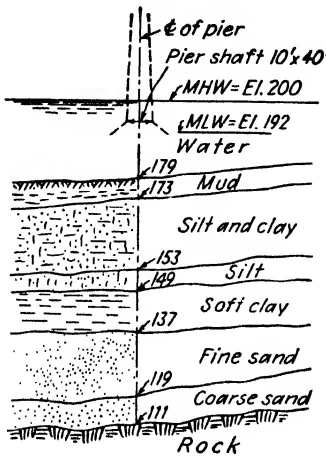


FIG. 12-29. Site for a bridge pier.

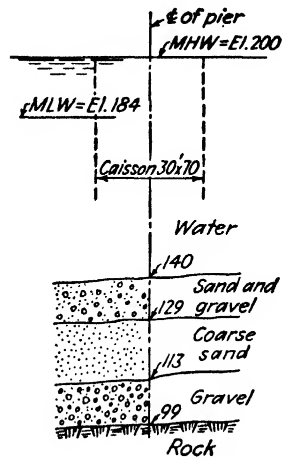


FIG. 12-30. Soil profile at site of a large bridge pier.

water. The shafts are 30 ft. c.c. with a connecting portal strut near their tops. The vertical load at the base of each shaft is approximately 1,500 kips. What is the best construction?

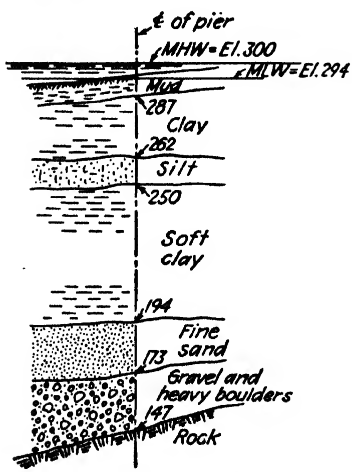


FIG. 12-31. Soil profile at site of a bridge pier.

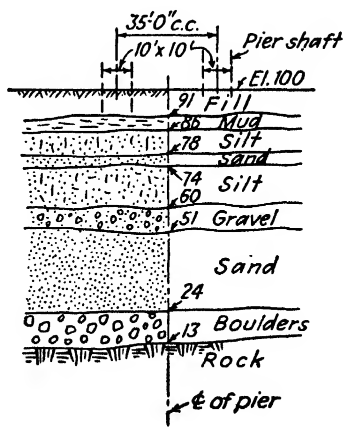


FIG. 12-32. Soil profile at bridge pier.

12-4. Plan the foundation for the pier indicated in Fig. 12-32 if the vertical load on each shaft at El. 100 is 2,000 kips. Notice that this section is taken transversely with respect to the bridge axis. Assume that ground water is ordinarily at El. 95, but occasional floods may reach El. 106. The site is at the bank of a wide sluggish inland river.

12-5. A main pier for a cantilever bridge across a navigable tidal river is to be built on the soils shown in Fig. 12-33, which is a view transversely with respect to the bridge axis. The pier is to have two large shafts on top of a solid base 15 ft.

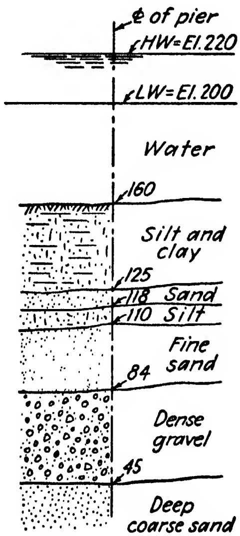


FIG. 12-33. Soil profile at site of a large bridge pier.

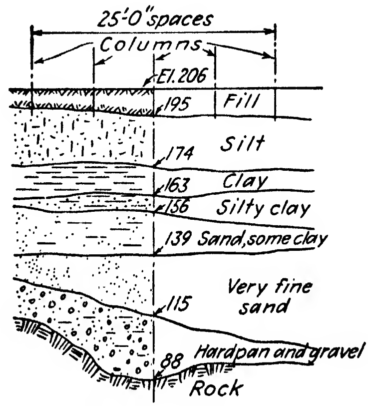


FIG. 12-34. Soil profile at part of site for large, multistory building.

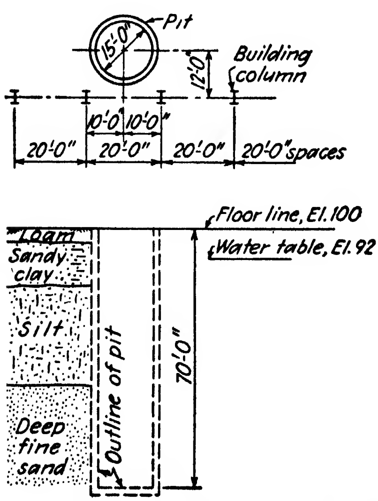


FIG. 12-35. A deep pit at an industrial plant.

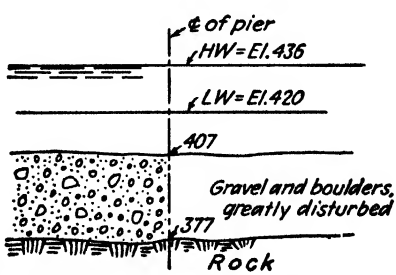


FIG. 12-36. Soil profile at site of pier in swift river.

wide and 45 ft. long with its top at El. 225. Assume the total vertical load at El. 225 to be 5,200 kips. Plan the construction.

12-6. A large, multistory office building is to be constructed near the water front of a large port city. The soil profile at a portion of one row of columns is shown in Fig. 12-34. Assume that the middle column shown has a total load of 1,350 kips. Design a foundation for it, and plan how to build it.

12-7. Figure 12-35 shows the soils and neighboring construction near the site of a deep pit needed for manufacturing processes at a new industrial plant. There is no superstructure resting upon the pit. The adjacent crane columns are each to support a vertical dead load of 120 kips and a live load of 230 kips. The pit will have pumps in the bottom to remove leakage water, but buoyancy must be resisted. How can the pit be constructed and the adjacent columns supported?

12-8. Figure 12-36 shows a transverse soil profile at the site of a bridge pier in a rapid northern river. The pier is to be a solid shaft 14 ft. wide and 45 ft. long at El. 420. The total vertical load at this elevation is 6,200 kips. Plan the best construction procedure for founding this pier upon the rock.

13

BRIDGE PIERS

13-1. Introduction. It is the purpose of this chapter to discuss some of the engineering problems that occur in the planning and detailed designing of piers for bridges and viaducts. In such planning, the method of construction—or at least one feasible method—may be the chief feature that influences the choice of the type of structure to be used below the surface of the ground or water. The character of the superstructure, its dimensions, and its weight and loads are also basic influences. Of course, the soil conditions at the site must be taken as they are and may greatly restrict the number of alternate practicable designs. Again, there may be special conditions that are of importance in a particular situation. Some of these will be described also.

This planning of the foundations is an integral part of the development of any major bridge project. The general features of the superstructure and of the substructure have to be worked out together in any actual case. Careful planning in advance may save many thousands of dollars, as well as avoid severe headaches for the engineers and owners.

In such major projects, safety of the structure is vital because life and tremendous property values are involved. Thorough investigations of the local conditions are absolutely essential, and so is careful study of all probable future developments that may affect the structure and its site. Except for certain temporary construction, the engineers who plan a bridge should look upon it as a structure whose useful service is to be almost eternal. Certainly, structural insecurity should not be the cause for abandoning or replacing an important structure. Obsolescence may develop because one can seldom anticipate all future developments. However, wise planning may minimize the probability of such an eventuality.

The principles involved in planning pier construction are to be illustrated rather than the vast number of details, although some of the last that may be useful will be shown also. Most of the problems discussed relate to bridge piers because viaducts and bridge approaches include similar but perhaps lesser difficulties.

13-2. Architecture. Cooperation of architects and engineers is extremely important in the early planning of a bridge and all its major parts so that the completed structure may present the most satisfactory appearance consistent with the service required, the local setting, the subsurface conditions, and practicable economy. Basic proportions, harmony of parts, consistency of style, suitability to its purpose, and obvious efficiency —these are generally the things that cause a bridge to present a pleasing aesthetic effect. Fancy details and gingerbread adornments are often both useless and incongruous. If the structure as a whole is not well suited to its site and service, minor details cannot camouflage this fact.

The preliminary studies of the topography, subsurface conditions, and superstructure are likely to suggest a few alternate possibilities for the bridge. These should be presented in perspective drawings as well as in projection so that all concerned with major decisions may judge the relative merits of the appearance of each suggested structure. The time and effort given to these studies are likely to be of great benefit.

An engineer should remember that the general public sees the structure as an entity in its surroundings, and they like it or they do not. Few are able to perceive the engineering problems involved, especially those connected with the subsurface work. Sufficient study will usually produce a design that fulfills the requirements of architectural and engineering suitability, and that attains optimum economy.

The general shape of a pier above water and its architectural features are largely dependent upon the type, size, and dimensions of the superstructure. Thus, it often happens that a specific type and size of pier are desired at a certain spot. It is the engineer's duty to try wholeheartedly to devise safe and economical means of supporting this structure at the place where it is wanted. Sometimes a careful study of the problems of safety and construction at this spot reveal so many difficulties and such expensive work that they react to modify the preferred general plan. Therefore, study and restudy may be necessary. The urgency to get work going should not induce one to fail to respect the importance and great value of the planning that leads to the best design.

The shapes and details of piers that may be used are almost infinite in number. Various illustrations are given to suggest ideas so that the reader may benefit by examining them when planning piers of his own.

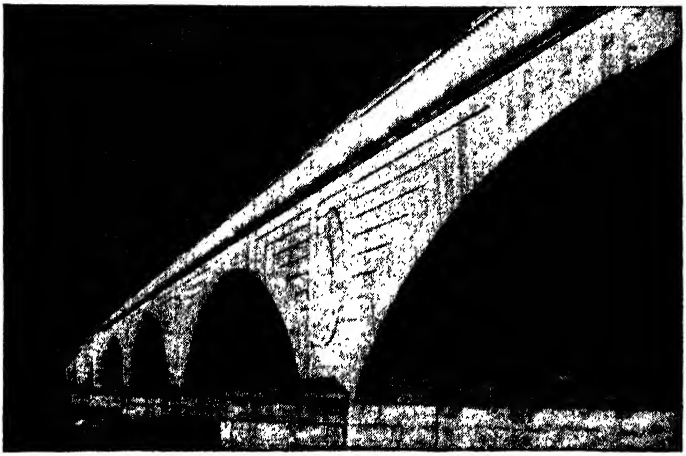
13-3. General choice of location. The alignment of a bridge may, for practical purposes, be determined by conditions beyond the structure. For example, in a densely built-up city, it may be almost necessary to place a bridge in line with a main traffic artery, because of the cost of real estate and the diversion of traffic if a new alignment were used. The foundation engineer then must meet and overcome the conditions that exist there if he can do so within reason.

In other cases, there may be a few neighboring sites that are possibilities. Such a condition may exist where a city on one side of a river wishes to build a bridge extending one of a series of parallel streets across the stream to facilitate traffic to a suburb on the other side. Then studies of each site may reveal data that render one of them preferable to any other. When a highway or railroad crossing over a waterway or other obstruction is to be made in rural territory, the range of possible sites may cover scores of miles. Then a very general survey may enable engineers to select one or more sites that are advantageous and should be studied in detail.

If the necessary data are not available, it is desirable first to obtain a general idea of conditions at a preferred alignment somewhat as follows:

1. Erect a few markers on both sides along the tentatively selected alignment.
2. By triangulation surveys determine the distances between two fixed points on opposite sides of the waterway if accurate maps are not available.
3. Obtain sufficient topographic data to enable the drawing of a map of a narrow strip of terrain along the entire alignment.
4. Take photographs of such features of the site as may seem to be useful to those who will study the problem away from the site.
5. Take a few soundings across the waterway and along the proposed center line, so that a profile can be drawn.
6. Study the general geological conditions at the site as far as can be done visually.
7. Obtain information regarding flood heights, current, ice, wave action, traffic on the waterway, and any governmental requirements for channel depths and clearances.
8. Obtain such detailed information regarding approach connections, real estate, traffic estimates, and subsurface conditions as may be available and essential.
9. Take a few borings along the center line to reveal the general nature of subaqueous soil conditions. These may be spaced at wide intervals and at such points as seem to be obviously desirable; *e.g.*, at edges of channels, at pierhead and bulkhead lines, and at shoals that are likely locations for piers and abutments.

From the preliminary maps and profiles, the first studies of the structure can be made. After the planning has progressed far enough to reveal what seems to be the best design, and after this plan has been tentatively approved by the responsible persons, then careful and detailed explorations of the soil should be made at the proposed locations of all piers and abutments. Sometimes, one boring and several undisturbed soil samples from it will be sufficient if taken on the center line of each proposed substructure. If the bottom is irregular, two borings at each such location may be enough, these being about 50 ft. apart and placed symmetrically about the center line of the bridge and along the transverse axis of the proposed pier. In other cases, it may be advisable to make at least one boring at each of



(a) Arlington Memorial Bridge at Washington, D.C.

(b) Cross County Parkway Bridge between Mt. Vernon and Yonkers N. Y., in the Fleetwood section.

FIG. 13-1. Two examples of multispan concrete bridges. Notice the details of the piers. *(Courtesy of the Lone Star Cement Corp.)*

the four corners of the pier. If the construction of a pier may involve sinking a caisson to rock, many more borings may be needed in a closely spaced checkered pattern over the probable bearing area. This may be desirable also when a pier or abutment is located in a present or former harbor frontage in order to explore for hidden obstructions. Such specific and adequate information not only helps to plan wisely but is likely to produce great economies by removing some of the uncertainties that would otherwise have to be covered by contingency allowances in a contractor's bid on the construction work.

Careful, fairly accurate studies of the most feasible sites for a large bridge project cannot be made hastily. Relative serviceability and practicability may have to be settled by expert judgment. The determination of relative economy generally requires the comparison of estimates based upon reasonably complete preliminary plans. A few thousand dollars spent in making these studies will usually save relatively large sums in the cost of the project.

It is sometimes said that the cost of substructure work of a large bridge project should approximately equal that of the superstructure. What is meant is that, if the cost of a single pier will be large, it is economical to use relatively long spans so as to minimize the number of piers. When the foundations are deep and the superstructure is at a high level, a pier is likely to be large, heavy, and costly in itself whether it supports a small load or a heavy one. However, it is best to plan a structure primarily to fit its site and for the service required. Local conditions generally are such as to make certain spots the natural or almost necessary locations for at least part of the foundation structures; others must automatically be avoided. The rest will usually work out to produce a plan that coordinates with these control points. Such points are the following:

1. Streets that must be cleared.
2. Real estate that must be avoided.
3. Real estate that should be utilized.
4. Channels that must be spanned.
5. Pierhead and bulkhead lines beyond which substructures must not project.
6. Rights-of-way of railroads, highways, power lines, and other utilities that must be avoided.
7. Islands and peninsulas that are situated advantageously.
8. Subsurface as well as visible rocky ridges or gravel bars that provide economical sites for piers.
9. Deep subsurface valleys, troughs of muck or silt, and lenses of weak materials that are undesirable as supporting structures.
10. Points that are automatically needed to produce symmetry of the superstructure. For example, a three-span continuous deck bridge may have the two inner piers located just outside of the channel lines. The pier or abutment at one end may be restricted to a certain point because of some local condition. Therefore, the one at the other end may necessarily be situated so as to provide a side span that is equal to the first one even

though the foundation conditions here are not so desirable as they are 50 ft. beyond this point.

Of great value is the ability to see the important features that affect a project, to devise the best plan or alternate plans quickly, to grasp a picture of the construction problems of the entire structure, to estimate the relative costs reliably, to weigh wisely the intangible values of one plan vs. another,

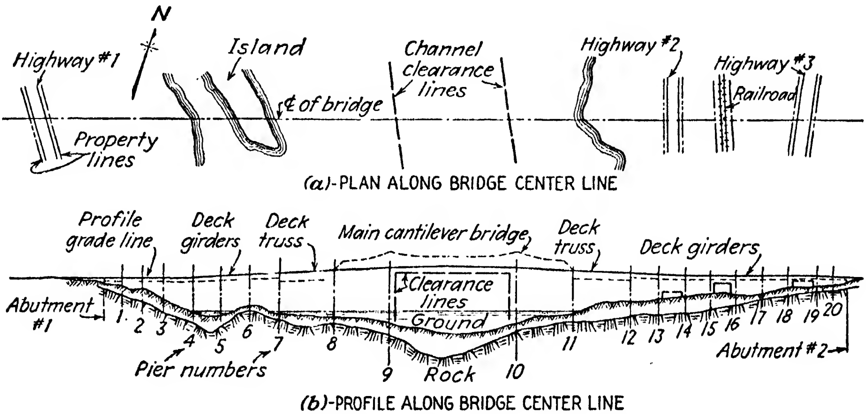


FIG. 13-2. Example of planning locations of piers and abutments of a highway bridge. Roadway is to connect to Highway No. 1 and to grade east of Highway No. 3.

and to make wise and prompt decisions. Such ability springs from a background of experience and study combined with common sense and excellent engineering judgment. It is not attained overnight.

13-4. Scour. The undermining of bridge piers by the scouring action of flowing water is one of the serious dangers to be guarded against when one plans such foundations. It is generally difficult to predict just what the future may bring forth in this connection. It is a subject that warrants careful study in each situation, because past failures of bridge piers and abutments from scour have been all too numerous and serious. There is much more to be learned about this phenomenon.

The required area of the waterway is not usually critical in the case of long bridges that have just a few tall intermediate piers, at least as far as the flow of water alone under the bridge is concerned. This is because such a bridge will generally have a superstructure with moderately long spans. However, the following are cases in which the provision of adequate and unobstructed cross-sectional area for stream flow during floods may be very important.

1. A bridge at such a low level that the superstructure may become immersed if the current is appreciable and flow is hindered. A minimum number and width of piers may then be advisable.

2. A narrow gorge containing a stream that is sometimes a raging torrent. Here it may be best to use no piers at all in the stream bed.

3. A small low bridge across an ordinarily dry stream bed located in an area where cloudbursts occur, such as in the Southwest. In such cases the

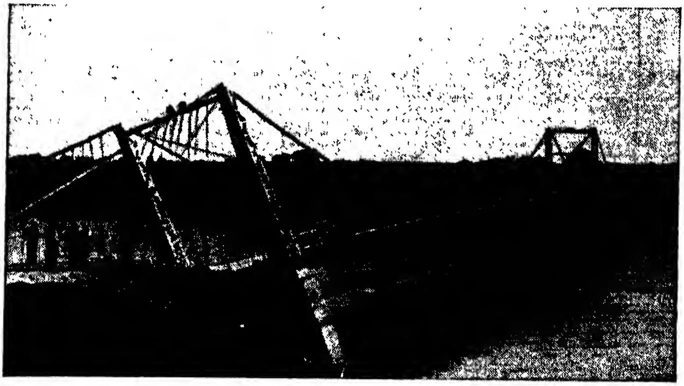


FIG. 13-3A. The Hidalgo-Reynosa Bridge across the Rio Grande, after one pier tilted because of scour. It was righted and supported upon new concrete piles. *(Courtesy of the Lone Star Cement Corp.)*

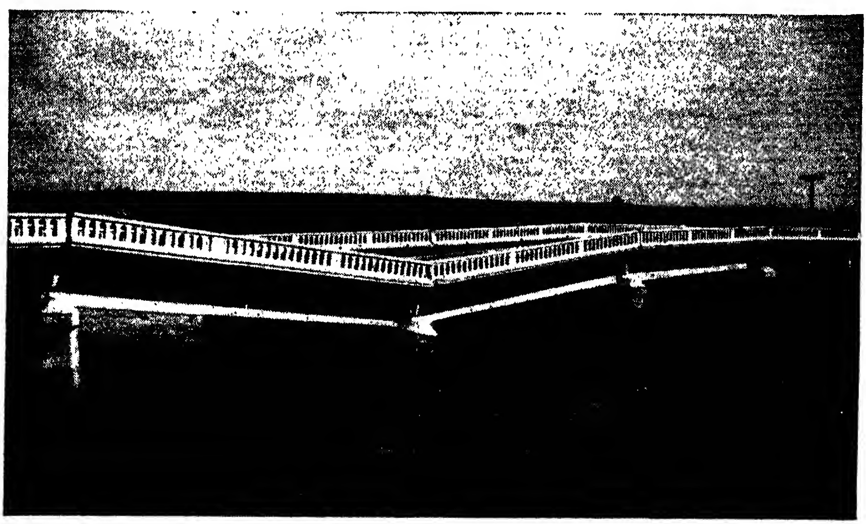


FIG. 13-3B. Santa Ana River Bridge in California after subsidence caused by scour during the floods of March, 1938. *(Courtesy of Ralph W. Stewart.)*

required area is indeterminate. The common use of paved sags in lieu of bridges for many highways in that region may be the best procedure, letting the stream flood over the road temporarily when a cloudburst occurs rather than building a bridge that tends to dam the valley.

4. A narrow tidal estuary used by boats where the accentuation of tidal currents and the presence of obstructions may be undesirable.

5. Streams that are filled with heavy ice cakes during the spring break-up.

6. Streams that are used for transporting logs in rafts or singly because the piers may cause log jams.

The determination of the best type and spacing of piers is a matter to be studied carefully for any particular site where the area of waterway is important. The answer is not readily obtained by substituting assumed

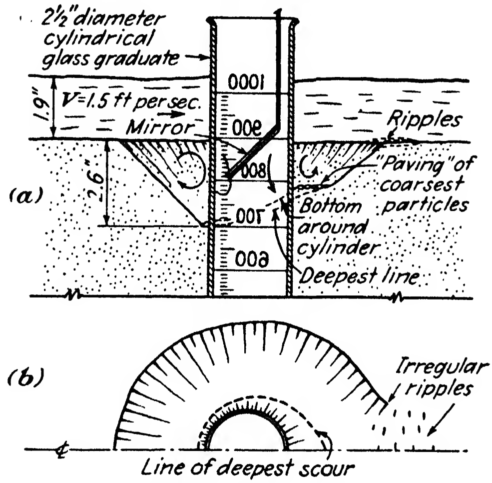


FIG. 13-4. One-liter glass graduate used as model of tubular pier to reveal character and extent of scour caused by flowing water. (a) Longitudinal section; (b) half plan. (Courtesy of C. J. Posey, State University of Iowa, Iowa City, Iowa.)

values in an empirical formula. Naturally, closely spaced piers tend to facilitate the jamming of driftwood and debris during a flood, and such jams may cause a more severe nozzle effect locally that speeds up the velocity and increases scour. It is necessary to make a special study of the flood records of the stream, to secure all available evidence of past flood heights, to determine the probable worst annual flood conditions of the future, to get a fair idea of the velocity of the current before and after the bridge is built, and to compare the estimated costs of various reasonable arrangements for the structure. With such data, the engineer must then use his best judgment in making a decision between conservatism at high cost on the one hand and economy with possible danger on the other.

It may seem that, if a river bottom is composed of muck or silt or even of fine sand, this is evidence that the currents at that site are too small to cause scour. Nevertheless, these fine materials may be disturbed to considerable depth, moved downstream during a flood, and then replaced by new and similar deposits as the current decreases. It is somewhat as though part of the bottom flowed like a muddy stream, especially close to the piers. Figure 13-4 shows some experimental data on scour.[1] One should remember, too, that fine particles are transported easily and that it may not require much increase of current to produce very noticeable effects in the deepening of the waterway.

One might think that a stream with a bottom of boulders and coarse gravel would not be endangered by floods. However, the transporting power of rapidly moving water in large volumes is tremendous. During a flood, boulders and gravel may be disturbed to considerable depth. As an extreme example, the gravelly bed of the Colorado River is apparently churned up or moved along at a depth of many feet during a major flood, this apparently being the cause of the grinding action that wears away the bedrock below and deepens the canyon.

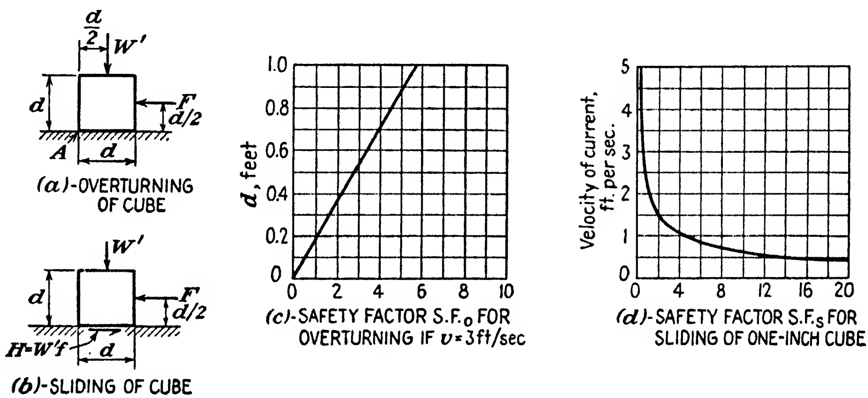


FIG. 13-5. Transportation of rock by flowing water.

Some qualitative data regarding the ability of flowing water to move solid particles may be obtained as follows, referring to Fig. 13-5:

1. Assumptions:

The weight of submerged solids = 100 p.c.f. (buoyant weight = W')
The weight of water = 62.5 p.c.f.

$$\text{Impulse exerted by flowing water} = F = \frac{W}{g}(v - 0) \qquad (13\text{-}1)$$

where F = force in pounds, W = weight of water impinging upon body per second, g = 32.2 ft. per sec.², and v = velocity of water in feet per second. This assumes that the water is deflected at 90° to its former path—more severe than the probable case.

Particle is a cube with side = d.

[1] C. J. Posey, Why Bridges Fail in Floods, *Civil Engineering*, February, 1949.

2. Rolling. As shown in Fig. 13-5(*a*), the body will tip about point A. Taking moments about A, the safety factor against overturning is

$$\text{S.F.}_o = \frac{W' \times \frac{d}{2}}{F \times \frac{d}{2}} = \frac{W'}{F} = \frac{100d^3}{\frac{62.5d^2v^2}{g}} = \frac{1.6dg}{v^2} \tag{13-2}$$

Assuming $v = 3$ ft. per sec. as a sample case, the curve in Sketch (*c*) can be plotted. This shows that, for a current of this magnitude, a cube having a side of approximately 2 in. would be at the point of rolling, safety factor $= 1$. A 1-in. cubical piece would start to roll in a current of approximately 2 ft. per sec.

3. Sliding. Figure 13-5(*b*) pictures the sliding conditions for a cube. For equilibrium,

$$H = W'f = F$$

Then, for this case,

$$100d^3f = \frac{62.5d^2v^2}{g}$$

$$\text{S.F.}_s = \frac{100d^3f}{62.5d^2v^2/g} = \frac{1.6dgf}{v^2} \tag{13-3}$$

This, compared with Eq. (13-2), shows that the body will slide before it will roll if f is less than unity, and a value of 0.6 to 0.75 may be assumed as a maximum.

4. Probable conditions. In most cases, the material in a stream bed is worn considerably so that the particles are somewhat rounded. The safety factor against rolling is then far less than the value in Eq. (13-2) because point A of Fig. 13-5(*a*) is much closer to the line of action of W'. If the lever arm is reduced, the safety factor will be decreased accordingly. The force F is probably decreased also. At least, one can see that small rounded particles may move rather easily in even moderate currents.

In Fig. 13-6, Mavis and Laushey[1] show the results of experiments and propose a formula for "competent" velocity, material in a stream about to move with the current. This, as they say, may "only *suggest* an answer." Alluvial beds like that of the lower Mississippi River may scour seriously with only a slight increase of current.[2] Erickson reported that an increase of 40 ft. in depth occurred alongside the 111- and 121-ft. sand islands during the construction of the 660-ft. main span of the bridge over the Mississippi River at Baton Rouge, La. This was in spite of the use of mattress work on the bottom.

[1] F. J. Mavis and L. M. Laushey, Formula for Velocity at Beginning of Bed-load Movement Is Reappraised, *Civil Engineering*, January, 1949.

[2] E. L. Erickson, Some Measurements of Velocities and Scour at a Mississippi River Bridge Pier, Public Roads Administration.

When the stream bed is composed of fine materials and when the current occasionally has considerable velocity, this matter of the obstructing effect of piers may be of great importance in disturbing the existing bottom through scour. As a qualitative example only, assume that it is proposed to build a bridge having a series of 100-ft. spans with piers 8 ft. wide at and below the water line. The original flood velocity V is 6 ft. per sec.

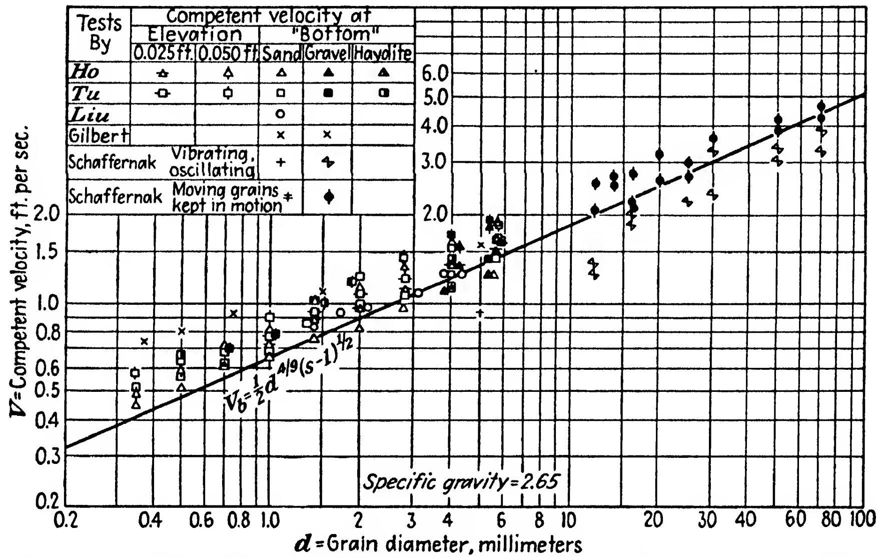


FIG. 13-6. Experimental data showing relation of size of grains and velocity of current to cause bed of stream to start moving. Also proposed formula to assist in predicting results. *(Courtesy of F. J. Mavis and L. M. Laushey, Carnegie Institute of Technology, Pittsburgh, Pa.)*

Neglecting variations in depth, the clear area of waterway A' is then $0.92A$, where A is the original area of a portion of the waterway 100 ft. wide. The effective area may be even less than this. The result will be a rise of water level upstream until there is sufficient head to increase the velocity under the bridge enough to maintain the required discharge Q. Then, approximately,

$$Q = VA = V'A'$$

The new velocity

$$V' = \frac{VA}{A'} = \frac{6A}{0.92A} = 6.5 \text{ ft. per sec.}$$

This is an increase of more than 8 per cent in velocity and may cause considerable local scouring.

The necessary depth of embedment of a pier supported directly upon ground depends upon the maximum current, not just upon the height of flood water. One cannot specify safely a set of dimensions for these embedments for any and all conditions. Such figures should be determined by the engineer after a thorough study of each case.

The bottom of a pier that rests upon soil rather than upon bedrock or deep piles should be far enough below the probable scour line to ensure safety. Yet who can be sure what distance is "far enough"? It has been stated[1] that, for the American Southwest, "scour extends two times flood water depth in the channel." Figure 13-7 and Table 13-1 show data reported by R. W. Stewart[2] after the storm of March, 1938. He states that the Atchison, Topeka, and Santa Fe Railway Co.'s policy is to keep its footings 25 ft. below stream bed, and that its record of safety is generally good. However, depths of 15 to 18 ft. in fast streams with gravel and boulders were not sufficient.

Excavation during construction may disturb the adjacent material sufficiently to enable the current to move it more easily and to produce a narrow channel alongside the pier as shown in Figs. 13-8(*a*) and (*b*). The water may even attack the soil under the base. The constriction of a waterway because of the presence of the piers, by its tendency to produce a nozzlelike effect, increases the velocity of the stream so that the erosive action evidenced in the past may not be the same in the future but may be more severe.

The action of water flowing past a rectangular pier is shown qualitatively and rather empirically in Fig. 13-9. The increased velocity of the water as it swirls and flows past the pier tends to wash away particles of the soil locally. The eddies caused in the stream tend to suck up particles from the bottom and carry them away. These particles may be deposited again in the slack water beyond the downstream end of the pier, but that is irrelevant. The removal of the supporting soil is the danger. Furthermore, the scouring away of material alongside the pier may cause shearing failure of the ground below the pier because of the lack of embedment. A layer of riprap dumped around the pier is beneficial sometimes. In other cases, scour may undermine the stones so that, eventually, they are not where they were originally placed and they may not offer fully the intended protection.

A very careful study should be made of the direction of currents. If the piers are skewed with respect to the direction of flow, as pictured in Fig. 13-10, their effective obstructing value is obviously increased from w to some amount more nearly equal to w_1. The reduced clear width w_2, therefore, tends to cause a larger velocity of flow and greater scouring tendencies, especially along sides A of the piers. Considerable suction from

[1] F. D. Hartford, Notes on Driving Timber Piles, *Engineering News-Record*, Nov. 29, 1945.

[2] R. W. Stewart, Safe Foundation Depths for Bridges to Protect from Scour, *Civil Engineering*, June, 1939.

TABLE 13-1. Data on Bridges Shown in Fig. 13-7

Item No.	Name and location	Type of structure	Length of adjacent clear spans in ft. at 90° to channel	Character of stream bed	Cause of failure
1	Cucamonga Wash Br., San Bernardino Co.*.	Concrete without piles	15	Sand and gravel	Scour
2	Malibu Creek Br., Coast Highway.........	Concrete without piles	30	Sand with some gravel	Scour augmented by drift and debris
3	Ventura Riv. Overflow Br., Coast Hw'y*....	Concrete without piles	28	Sandy gravel	Scour
4	San Antonio Creek Br., 5th St., Pomona*....	Concrete without piles	27	Sand with some gravel	Scour
5	Huntington Dr. Br., Eaton, Wash...........	Concrete without piles	20	Gravel wash	Scour
6	Pac. Elec. Ry., Glendora line, San Gabriel Riv.	Concrete without piles	50	Heavy gravel and boulders	Scour
7	Santa Ana Riv. Br., Coast Hw'y, Orange Co...	Concrete with piles	60	Dune sand	Scour
8	Whitsett Ave. Br., Los Angeles Riv.........	Concrete with piles	40	Hardpan	Scour
9	Southern Pac. R.R. Br., Los Angeles Riv......	Steel plate girders, concrete piers, with piles	70	Gravel with some boulders	Scour
10	Lankershim Blvd. Br., Los Angeles Riv.*.....	Concrete with piles	40, 20	Sand and gravel	Scour somewhat augmented by debris
11	Santa Clara Riv. Br., Bardsdale, Calif.*....	Reinforced concrete and steel truss, concrete piers, with piles	129, 30	Sand and gravel	Scour materially augmented by debris
12	Santa Clara Riv. Br., Saticoy, Calif.*.......	Steel truss, concrete piers with piles	129	Sand and gravel	Scour materially augmented by debris
13	Santa Ana Riv. Br., 4 Mi. So. of Yorba Linda	A-frame trusses on concrete pile bents	40	Sand and gravel	Scour materially augmented by debris
14	Santa Ana Riv. Br., Norco, Calif...........	Timber trusses, steel cylinder piers, piles	46, 60, 120	Sand and gravel	Scour materially augmented by debris
15	Sespe Overflow Br., Ventura Co. Road to Castaic	Timber stringers on concrete framed bents	30	Sand, gravel, and boulders	Failure due principally to debris
16	A.T. & S.F. Ry. Br., L.A. Riv. near No. Broadway	Steel trusses and plate girders	30, 80	Sand, gravel, and boulders	Failure due principally to scour

* Water rose above bottom of deck.

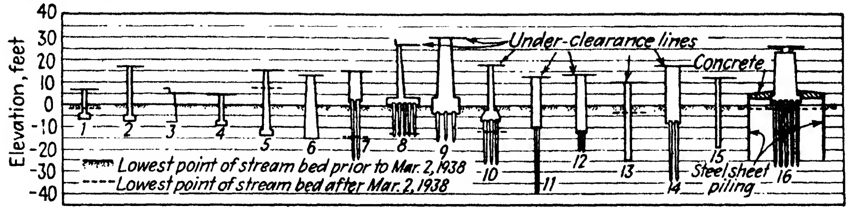


FIG. 13-7. Diagrammatic cross sections of piers and abutments that failed because of scour in the flood during March, 1938 in California. (Courtesy of R. W. Stewart.)

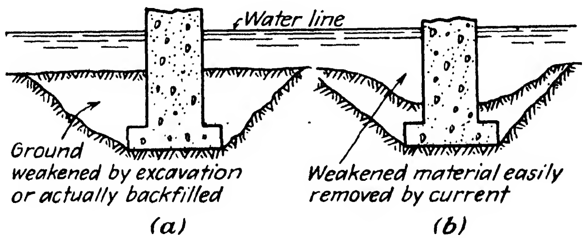


FIG. 13-8. Scour at a pier.

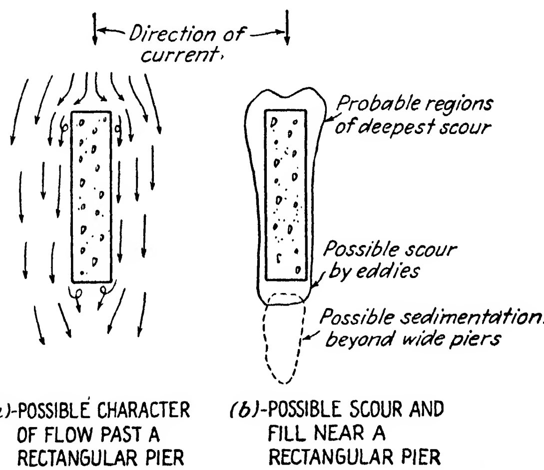


FIG. 13-9. Possible action of water flowing past a pier.

eddies may also exist along sides *B*. In severe cases, it may be wise to use skewed spans for the superstructure and so minimize the interference of the piers with flow.

Another matter for study is the shifting of the channel in a stream that is inclined to meander so that the channel now is not where it was 10 years ago, and it may be in still another position 10 years hence. Winding streams in wide flat valleys are likely to do this. Such action may seem to be too slow to worry about. Nevertheless, the life of a bridge is planned to be 50 years, 100 years, or more. Much may happen in such periods. Bars that deflect the stream now may erode away, and others may appear in

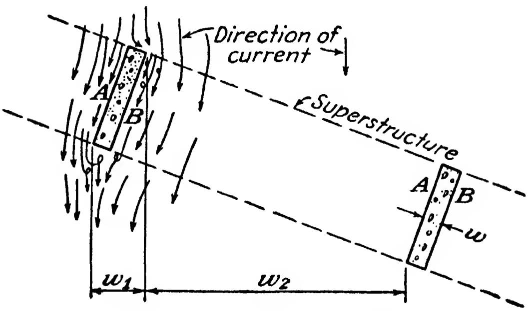


FIG. 13-10. Flow past skewed piers.

new positions so that the deep channel will migrate laterally. It is difficult to predict these things, but they should be guarded against as far as practicable. The Mississippi River is an excellent example of this action.

An important detail in the planning of piers is streamlining. If the upstream end of a pier is made with a cutwater, as pictured in Fig. 13-11(a), this reduces the suddenness of the deflection of flow that occurs with a square upstream end. The increased velocity from point A along the sides will probably cause some scouring. If the downstream end is square, the eddies beyond B are still harmful. A cylindrical shape at both ends, as in (b), is fairly effective when the current is not very rapid. The most effective shape to minimize resistance to flowing water is more like the streamlining shown in (c), but this refinement is seldom necessary.

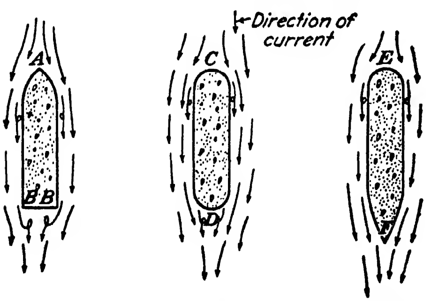


FIG. 13-11. Streamlining of piers.

It may seem desirable to use pile fenders or submerged sheet piling placed upstream from the pier, as shown in Fig. 13-12. This is done with the idea that these obstructions will prevent loss of the soil around and under the pier itself. However, the following comments should be noticed in connection with each of the following sketches in this illustration:

(a) If there is a small group of piles as shown here, the eddies behind the piles and the current along the sides of the pier may cause as much scour as though the piles were not used. Furthermore, if the piles are not close together, the speeded-up flow between them may cause severe local scouring.

(*b*) The wide sheet-pile fender shown here increases the effective obstruction to flow. The violent eddies at the upstream corners and the rapid inflow against the sides of the pier are likely to be dangerous.

(*c*) In this case, the sheet piling has been carried downstream as a continuous barrier slightly beyond the pier. It may even enclose the downstream end. The result is that the effective obstruction to stream flow is increased from w to w_1. If the piling is not close to the pier, overtopping

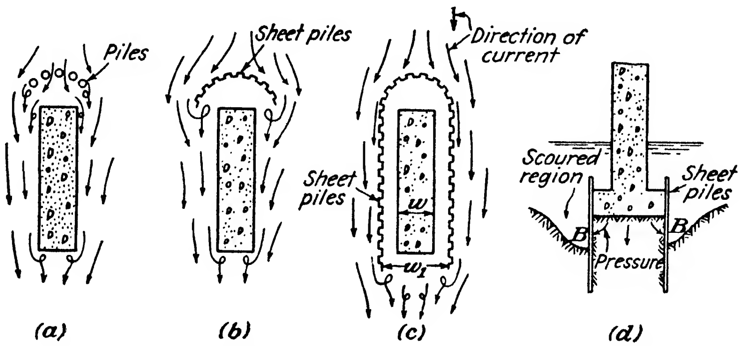


FIG. 13-12. Flow past piers with fenders.

and eddies may remove some of the intervening soil. Hence this space should be filled with riprap or a concrete pad when the velocity of flow is considerable.*

(*d*) This vertical section through the pier in (*c*) shows how failure may occur even though the footing of the pier is poured against the sheet piling. Scour that occurs along *B* may remove the lateral support for the piling so that the latter will yield and cause the pier to settle. Rods across the base to tie the piling together may resist this action only until they rust off. It is probably better and cheaper to deepen the pier itself.

Beach erosion may also be a problem in some cases. This applies mostly to the design of foundations for water-front structures other than bridges.

Long piles under a pier founded upon fine sediments usually prevent dangerous scouring because they necessarily penetrate to considerable depth. However, piles are expensive. When are they necessary, and when not? Answering this question is one of the responsibilities that the engineer must assume.

In the whole problem of bridge foundations—both piers and abutments—this matter of scour is one to be treated with great respect and care.

13-5. Wave action. Wave action against large bridge piers is seldom serious although it may be very troublesome in its effect upon cofferdams and caissons during the construction of those piers. Pile trestles and cause-ways having short spans and with superstructures near the water level are much more vulnerable to damage from wave action during maximum floods, high tides, and hurricanes. These structures are light, and waves that strike them may impinge against relatively large areas so that the resultant

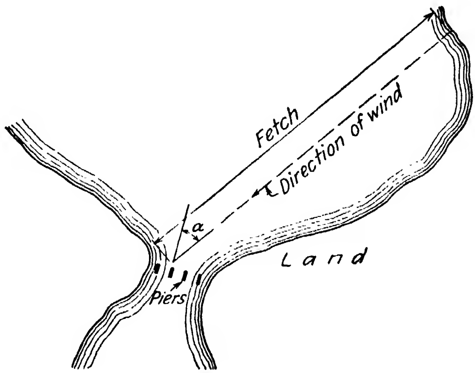
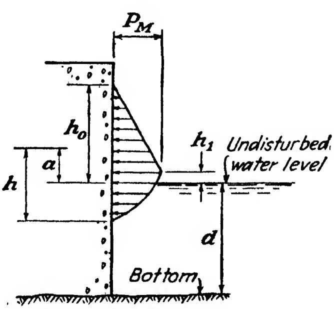


FIG. 13-13. Open water for estimating force of waves.

forces may be serious. Providing a little extra height of structure and strength of foundations may be good engineering.

The greatest exposure will probably occur when the wind is blowing at an angle with the axis of the pier, as indicated in Fig. 13-13. The compo-



Wind velocity, m.p.h.	h_o, ft.	h, ft.	a, ft.	h_1, ft.	Maximum pressure P_M, p.s.f.			Total pressure, lb./lin. ft. of wall		
					$d = 20'$	$d = 30'$	$d = 50'$	$d = 20'$	$d = 30'$	$d = 50'$
30	10.0	9.3	5.0	1.1	1,700	2,100	2,500	13,800	17,000	20,200
70	16.6	14.2	8.3	1.9	2,100	2,200	2,500	26,200	27,500	31,200
100	21.0	17.0	10.5	2.0	2,200	2,300	2,500	33,500	35,000	37,100

FIG. 13-14. Estimated values of maximum normal pressure caused by wave action. Assumed fetch = 100 miles. Based on formulas given by Paul Andersen's "Substructure Analysis and Design," Irwin-Farnham Publishing Co., Chicago, 1948.

nent of the wave impact parallel to the bridge may then exert an important longitudinal force along the bridge axis. However, the *fetch*—distance or length of clear water over which the wind blows—is usually small when the angle a in Fig. 13-13 is large because bridges are built across the narrowest portions of waterways. Furthermore, the wave moves along the pier instead of striking all of the side simultaneously.

The real magnitude of the impact produced by a wave striking a rigid obstacle is not known. A breaker hitting a sea wall evidently exerts a much larger force than does a roller hitting a pier in rather deep water. The magnitudes of the wave forces shown in Fig. 13-14 are empirical, but they give the engineer something to use in the comparison of designs. In using these data, the tabulated forces may be reduced to about 50 per cent for triangular or rounded ends of piers when the wind is perpendicular to the bridge. The same reduction may be used for winds at 45° to the superstructure, if the assumed force per linear foot is applied to the projected width of the pier normal to the wind direction.

13-6. Ice. In some places ice may have a real influence upon the planning of bridge foundations. This may be important in the location of piers, the magnitude of their spacing, and the detail design for their stability.

The lateral pressure of static ice freezing upon a pond or lake is seldom troublesome because it is usually balanced—if it exerts any real sideward pressure at all—because it does not move horizontally. Thick ice frozen to the piles of a trestle or the bents of a causeway may produce uplift when the water rises or, more particularly, extra downward load when the water level drops. However, the weakness of ice in resisting the bending caused by bulging upward or by sagging will generally cause it to crack so that there is little harmful effect, even though its adhesion to the piles is not broken.

Ice in motion is another problem. Anyone who has seen the spring floods in the St. Lawrence River or the breakup of ice in some other large northern stream will have considerable respect for its power. The author has seen ice form great jams in which large cakes were heaved as though shoved around by subaqueous giants. Finally, the jam would break up with a terrific rush and grinding. How can a bridge pier be designed to resist such forces, and what are their magnitudes?

A force equal to the crushing strength of ice, even at something like 1,000 p.s.f., when applied to the upstream end of a deep-founded pier is bad enough. The impact of a large floe colliding with the pier is still worse. And the wedging effect of ice jammed between two piers, or the river bank and a pier, may be extremely serious. Either the piers must be kept out of the dangerous portions of the river by the use of a long span or else some means must be devised to ensure the safety of the structure.

FIG. 13-15A. Air view of bridge over Indian River Inlet near Rehoboth, Del., subjected to pressure of drifting ice. Portion collapsed when a truck and an auto were passing over it, killing three persons and injuring two. *(Courtesy of Acme Photos.)*

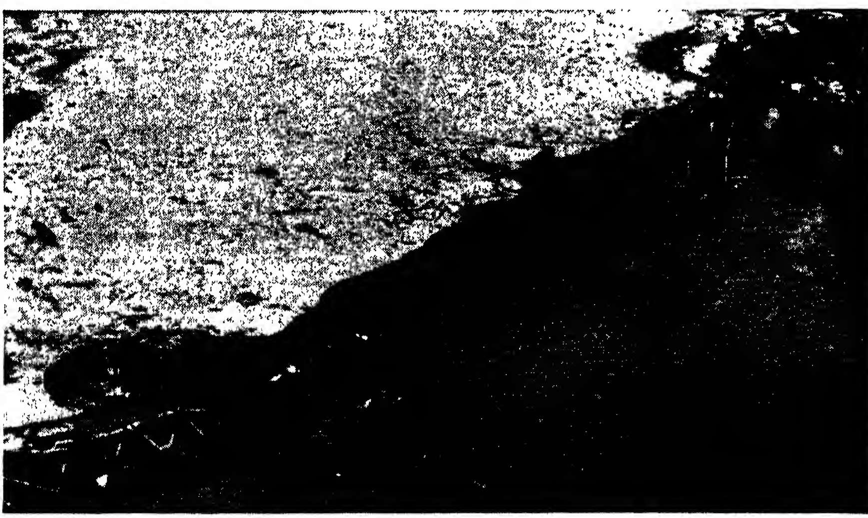


FIG. 13-15B. This ice jam in the Des Moines River destroyed this three-span bridge near Chillicothe, Iowa, in the spring of 1949. The water carried the center span down the river. The south (near) span later collapsed. The north span buckled but remained fast to its piers. *(World Wide Photos.)*

Fortunately, rapidly flowing rivers are generally shallow, and rock may often be within practicable reach. A pier then is to be designed not merely as a support for the vertical load of the bridge but as a rigid streamlined mass that can withstand these other forces. Resistance to sliding may be obtained by keying the structure to the rock; resistance to overturning downstream, by sufficient mass and length; and resistance to lateral tilting by mass and width of base below water.

Of course, a narrow cutwater at the upstream end of a pier is advantageous, but not so much so as one might think. *A starling* is one of the best devices to use for severe conditions. As indicated in Fig. 13-16(a), the cutwater is sloped as an inclined wedge or it may be on a curve as in (b). The ice then tends to slide up the starling until one edge is clear out of water and its weight is sufficient to break the cake over the "knife-edge."

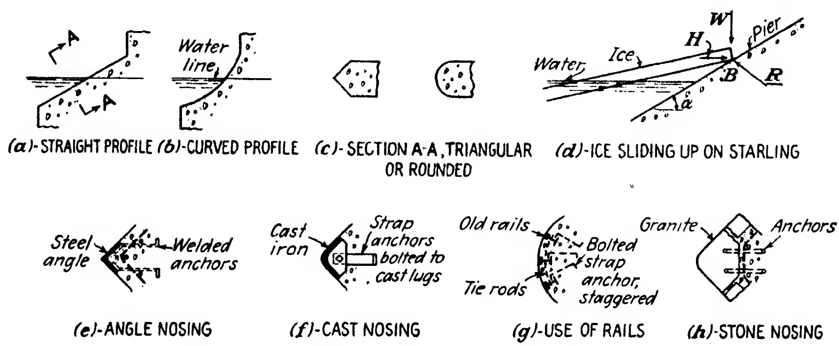


FIG. 13-16. Details and action of starlings as icebreakers.

The current then carries the pieces past on both sides of the pier. Sometimes the cutting edge of a starling may be shod with a heavy steel angle or with old rails, or it may be made of rounded stones or concrete. What is desired is sliding and breaking, not cutting of the ice. Of course, the position and slope of the starling are to be determined in accordance with probable flood levels and ice conditions at the particular site concerned. Plenty of length and a sufficiently flat slope for the starlings may cost some extra money, but the owners of the structure will be grateful when they see them function safely during some spring when the river is on a rampage.

There may be places where ice (or log) jams are likely to occur in spite of the starlings and piers that are as narrow at and above the water line as it is practicable to make them. This is a difficult problem to handle. Stone-filled cribs or concrete blocks with starlings may be located upstream from the piers in the hope that they will break up the ice before it reaches the bridge, or that they may cause the ice jam to occur between them rather than at the piers.

Drifting ice in large northern lakes and in tidal estuaries may also be troublesome. This may not justify the use of starlings but should be considered in the planning and design of the piers. The following illustrates how such ice may act.

There is a lake about 5 miles long in a north and south direction and 2 miles wide. In the spring floods, the water level rises and the ice tends to break into large cakes before melting. These are driven by the wind like large floes. If a northerly gale blows for a day or two at the right time, the ice moves toward the south. If the wind then shifts to the south, the ice drifts back to the north shore. Although the motion is slow and the ice may be somewhat "rotted," the author has seen huge cakes 12 to 18 in.

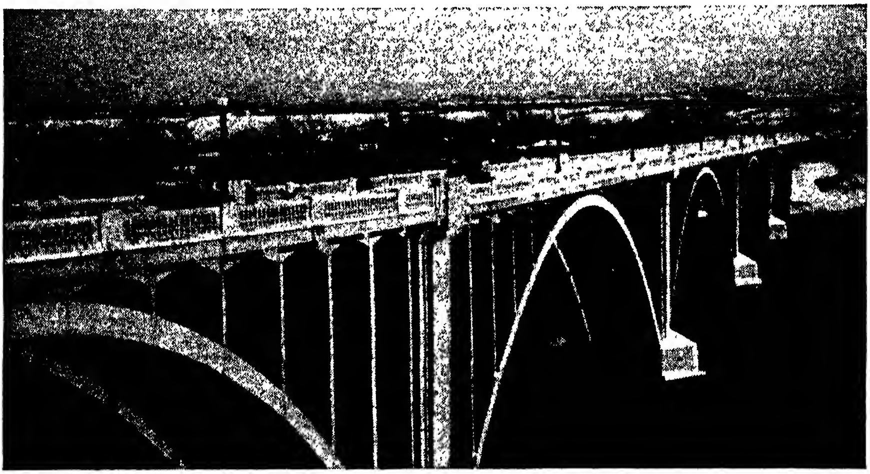


FIG. 13-17. The Pie IX Boulevard Bridge over the Des Prairies River in Montreal, Canada. The arch ribs are box sections of steel painted gray and sprayed with white sand. The spandrel columns are structural steel embedded in concrete, and the deck is reinforced concrete. Notice the design of the piers to resist ice and the current of close to 5 m.p.h. The greatest water depth varies from a low of 17 ft. to about 25 ft. at flood. Two of the piers were built with compressed-air caissons; the other three with open caissons. The bridge was built for the Province of Quebec by the Dufresne Construction Co., Ltd., Montreal. S. A. Beaulne was consulting engineer and the Dominion Bridge Co., Ltd., subcontractor for the steelwork. *(Courtesy of the Dufresne Construction Co., Ltd.)*

thick shoved over a shoreside highway embankment and some 50 to 100 ft. beyond it, forming heaps of ice cakes.

When a highway bridge was planned across an arm of this lake, a design for the piers was made with this ice condition in mind. The piers were to be founded upon piles in a mucky bottom under 30 ft. of water. A cheaper design was finally built, consisting of slender piers on piled footings.

One spring, some years later, it was discovered that the ice had tilted one pier by wedging action or diagonal drifting so that the bridge was endangered seriously. To remedy the situation (at a cost of many thousands of dollars) extra piles were driven beyond both ends of the pier, steel girders were erected between caps on these piles, and all were concreted against the old pier, thus in effect holding it up "by the neck." The bearings were then reset and the bridge was saved.

In most such cases where drifting ice is a hazard, a pier can be made long enough (perpendicular to the bridge) and heavy enough so that it can withstand the pressure applied at one end. It may be difficult, however, to resist the thrusts from wedging or from ice moving diagonally with respect

(a) The ice beginning to break up at 8:50 A.M., Mar. 27, 1947. The action of the pier nosing is shown clearly.

(b) General breakup of ice at 10:10 A.M. Pier No. 1 is in the foreground under the pile-up of ice.

FIG. 13-18. Piers for the Garrison Dam construction bridge across the Missouri River near Bismarck, N.D. These are founded upon 24-in.-diameter, $\frac{3}{4}$-in. welded-steel tubular piles. They have safely withstood the tremendous pressures caused by ice as thick as 3 ft. *(Courtesy of Frank Harrison, vice president, Missouri Valley Constructors, Inc., who built the piers, and E. E. Howard of Howard, Needles, Tammen & Bergendoff, Kansas City, Mo., who designed them.)*

to the bridge because the piers and their footings are usually relatively narrow in this direction. The plans for the structure may be made purposely, by the proper use of fixed bearings, so as to have the bridge itself act as a strut to prevent tipping of the piers in the direction of the bridge. Some possible arrangements are shown in Fig. 13-19(*c*). Of course, the piers should be sufficiently tall and slender so that they will deflect easily with thermal and live-load deformations of the superstructure, as indicated in the sketch. The bearings should be designed to resist the necessary shearing and overturning tendencies. If the structure is arranged as in Fig. 13-19(d), the piers are tied together by the central span, but wedging at one side might tip them until the superstructure bumps into an abutment, causing a longitudinal pressure in the superstructure itself.

When an engineer who is not a resident of the locality is studying the ice conditions at a particular site he should consult the people who have lived

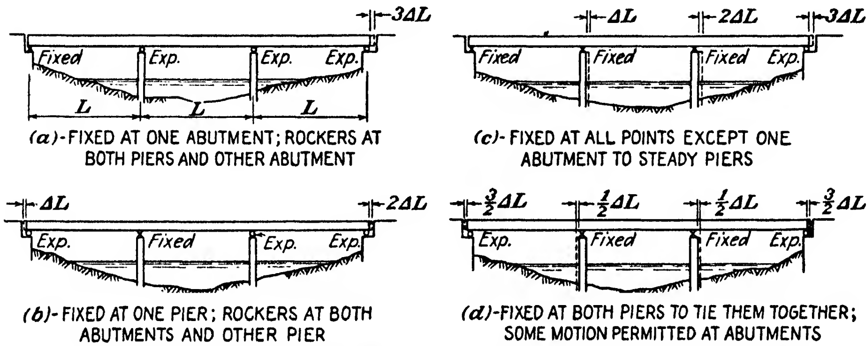


FIG. 13-19. Illustration of some arrangements for bearings of three-span continuous bridge. Movements shown for rise in temperature.

there for many years. He may also obtain data from the Weather Bureau and local town and county officials. The position of driftwood, scraped marks on shore and on trees, and damage to any boat docks and bulkheads nearby will sometimes yield useful information.

The following data are given only to assist the designer by making suggestions regarding magnitudes to be used for the horizontal forces from ice pressures in rivers. Each engineer is to judge whether they are inadequate or too severe for his own case because they are entirely empirical:

1. Crushing of ice against end of pier. Pressures per horizontal foot of width of upstream end of pier at water line, as given in Fig. 13-20.

2. Drifting lake or sea ice moving at angle β with long axis of pier. Total force P, 1,000 lb. per horizontal foot of projected width of pier at water line, as shown in Fig. 13-21. Notice that this width in sketch (a) includes the side BC as well as end AB even though the ice may tend to skid off the former. Furthermore, in (b) the width used generally should be w_t instead of $2w$ because of the jamming of ice cakes unless the columns are very far apart. The same principle applies for the row of piles in (c).

3. Ice jams. Equivalent to static hydraulic head of 5 ft. uniformly across stream applied to a length equal to the average of the span lengths supported by the pier being considered. If this is less than twice the pressures given in Fig. 13-20, use double the values given by this diagram.

4. Forces on starlings. Forces as in Fig. 13-20 for the corresponding rounded or triangular ends times the sine of the slope angle α, Fig. 13-16(*d*),

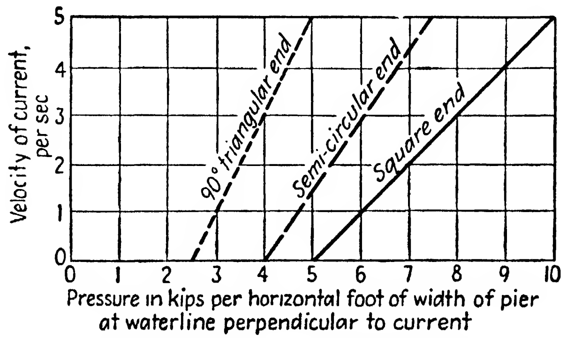


FIG. 13-20. Empirical values for pressure of ice against end of a pier.

of the starling with the horizontal. No reductions are to be assumed for item 3.

13-7. Choice of type of bridge and pier. As in many other engineering works, the type of structure to be used for a specific bridge depends upon many things besides numerical calculations. Usually the architectural character, the basic engineering principles of the design, and the general

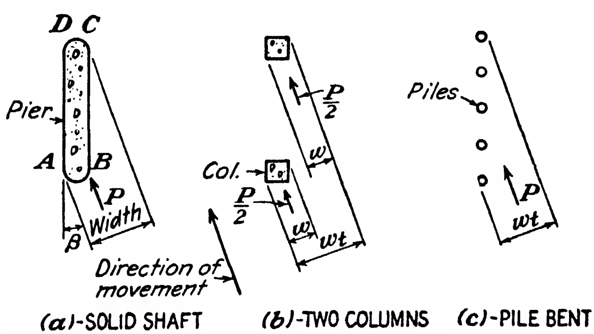


FIG. 13-21. Pressure areas on piers caused by ice drifting diagonally in a lake or other open water.

proportions and dimensions are determined rather specifically at first. Next, the general plans are made. Detailed studies and calculations then follow to show whether the desired structure will be safe, to prove that the plan can and should be built, and to establish all dimensions and details for contract and construction purposes.

From among all the things that could be done, how does one choose the best plan for a particular bridge? In practice, the planning of the foundations—piers and abutments—and of the superstructure constitutes one general problem, with each part influenced by and somewhat dependent upon the others.

First of all, the bridge must have a certain capacity and strength to accommodate and support the desired traffic. Next, the entire structure should be the most suitable, economical, and attractive one that the planners can conceive. This problem of the general planning of bridges is, in itself, an appropriate subject for a whole book. The problem to be dealt with herein is primarily that of planning the foundations. A few of the major things that may influence the choice of the basic features of the structure as a whole are the following:

1. Large vertical and horizontal clearances for shipping may so affect the planning that only certain types of high long-span structure are feasible, such as suspension bridges, arches, or cantilever and continuous trusses.

2. A high-level structure with uninterrupted traffic may be desired so that the use of a few tall foundations and long spans is obviously desirable, such as cantilever and continuous trusses or girders, arches, or perhaps a series of simply supported trusses.

3. Long high approaches may be so costly and undesirable that a low-level structure with some type of movable span is preferable. Then a series of short girder or truss spans may be the most practicable with a vertical lift, bascule, or swing bridge as the dominant unit.

4. Deck bridges afford a better view of the scenery than do through structures, especially for highway traffic. This may be one of the important starting points of the planning. Furthermore, if vertical clearances are not important, deck trusses, arches, or girders are generally more economical than are through ones having the same span lengths. This is due to economies in the superstructure as well as to the smaller height of piers.

5. The use of steel, concrete, or wood, if the use of one of these materials is necessary or highly desirable, will automatically affect the planning because each material is particularly adaptable to certain types of structure.

6. Rocky terrain with sound bedrock at shallow depths may permit the use of many types of short-span structure or of heavy long-span ones. Soft soils and shallow depths may render light short-span girders or trestles most desirable. Deep foundations may be individually so costly as to compel the use of a long-span superstructure. Deep rocky ravines may be ideal for arches. Thus, the topographic and soil conditions at a site may practically limit one's choice of structure to two or three general possibilities.

7. Foundations on weak soils may render the use of statically determinate structures preferable to arches and continuous bridges because of the effect upon the stresses in the superstructure if unexpectedly large differential settlements should occur. However, one should not build bridge foundations that are expected to be unreliable.

8. The amount of available funds may control much of what an engineer can do, but this should never be permitted to result in construction that is not safe. Matters of finance may compel one to build a series of steel truss or girder spans instead of a monumental structure of beautiful arches; or to build a simple low-level bridge with a movable span instead of a more impressive high-level structure. However, if a certain type of structure is needed, proper planning will usually enable one to produce an attractive structure at practically the same cost as a similar but homely one.

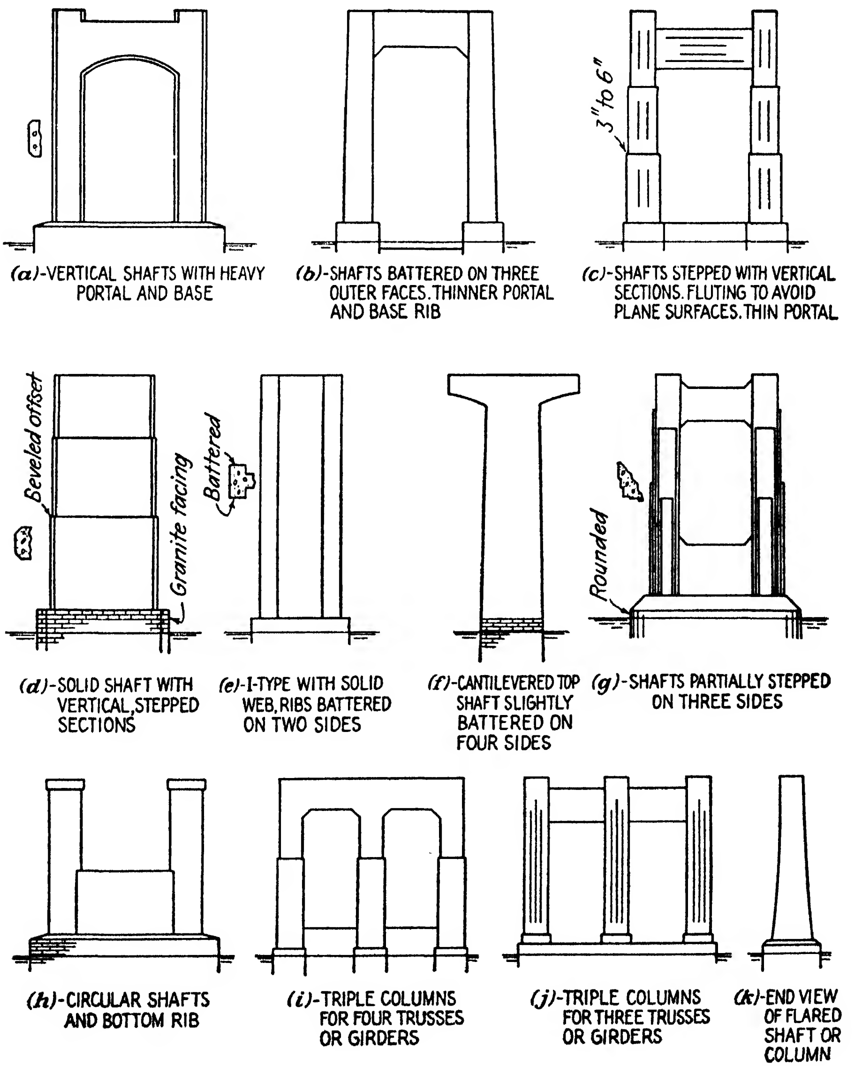


FIG. 13-22. General shapes of some piers for large bridges.

9. The type of traffic may limit the freedom of choice somewhat. For example, railroad service requires a stiff structure so that heavy steel trusses are more advisable than a flexible suspension bridge.

10. The personal preferences of the owner, of the architect, and even of the engineer himself may be of vast importance when a choice among types is made. What looks well to each individual? What seems best in his judgment?

There are many ways in which the type of superstructure to be built will either control or suggest the type of pier construction to be used, at least for the portion above water or ground. In order to illustrate some of the many varieties of bridges and to suggest types of construction for both superstructures and substructures, an extensive list of photographs is included herein. The captions accompanying them are to be treated as part of this text. The descriptions are made in sufficient detail to give the reader some idea of the reasons behind the use of that particular construction, and to show its advantages.

There may be situations in which the type of pier to be used is practically dictated by soil conditions and construction procedures. The problem that is usually presented to the engineer, however, is more likely to be stated thus: "This is what I want. How can it be supported safely? What will it cost? How can it be built?" However, when he finds that the proposed structure will be impracticable, uneconomical, or unsafe, it is his duty to make this clear to all concerned and to do his utmost to assist in the development of the most feasible plan.

A pier should have sufficient area at its top to receive the bearings. It should appear strong rather than weak or flimsy. It should be capable of supporting the lateral and longitudinal loads as well as the vertical ones. It should be practicable and durable, requiring a minimum of maintenance. And it should be attractive.

In general, a bridge with two trusses or girders is well adapted to support upon a two-column type of concrete pier; a wide bridge with three or four deck trusses or girders may best be held by a corresponding number of pier shafts; a superstructure of closely spaced concrete or steel girders may be placed upon a solid pier, a two-column and portal type, or a series of columns; where mass is required, a solid shaft may be used regardless of the number and position of the bearings.

The preceding statements undoubtedly still leave the reader uncertain as to how to proceed in the creation of a plan. Observation of existing structures, study and experience in bridge design and construction, ability in the making of freehand sketches or more elaborate drawings to picture various possibilities, and the development of some aesthetic and engineering sense to judge the merits of these possibilities—all these are helpful, and some are developed largely from practice.

Perhaps the following story is relevant. A young graduate engineer asked a consulting engineer of vast experience in bridge construction how the latter created a plan for a major bridge. The older engineer replied, "I do not know how I can tell you but, if you will come to work with me for ten years, then I shall not need to tell you."

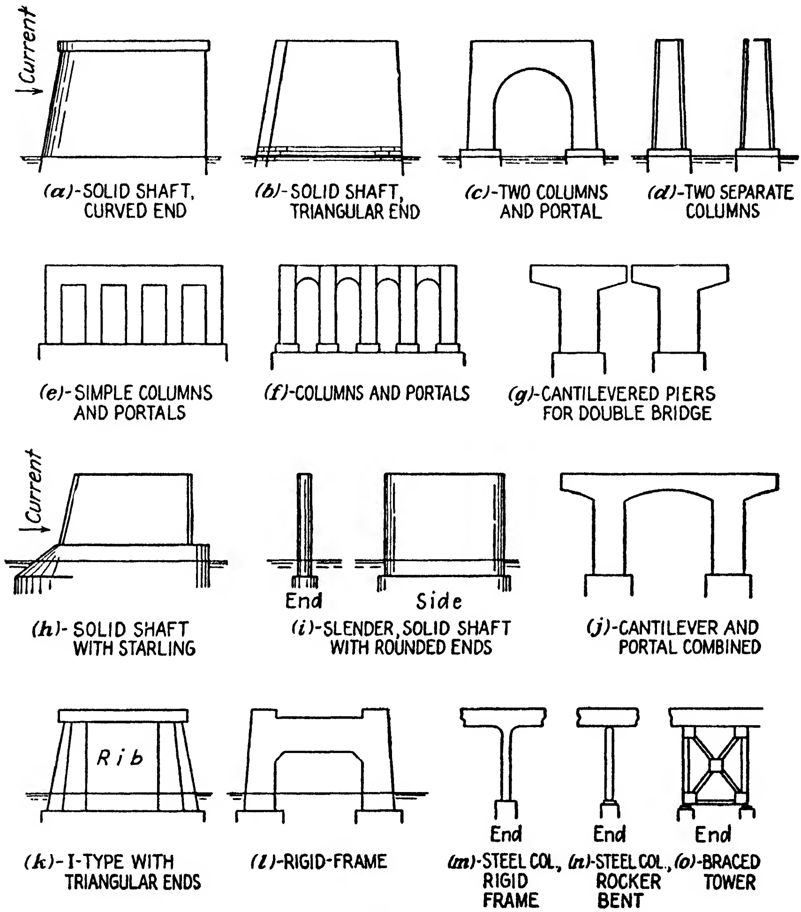


FIG. 13-23. General shapes of some piers for small bridges.

Nevertheless, much can be learned from study. In Figs. 13-22 and 13-23 are shown several types of pier. The drawings are made to picture the basic characteristics of the visible portions of the structures and to show various details rather than the methods of support below water or ground. In general, a pier is most satisfactory if it is simple, neat, and obviously appropriate. Perhaps this will be evident if the reader will study the structures shown in Figs. 13-24A to 13-31.

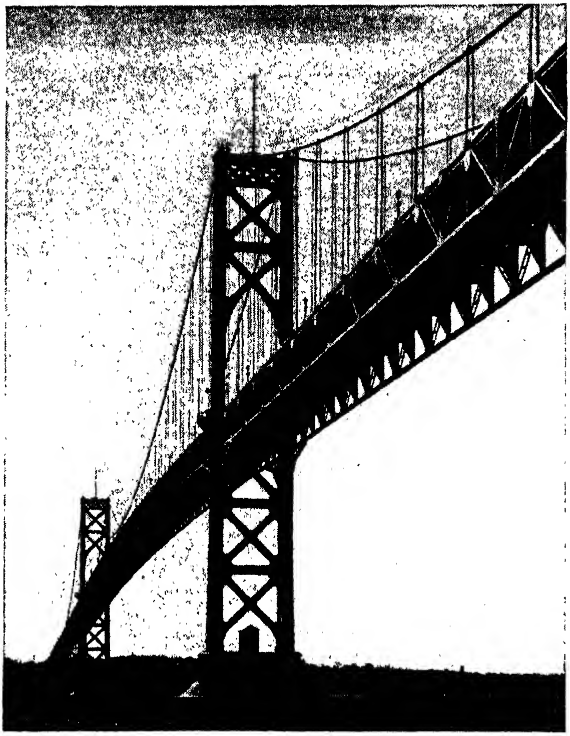


FIG. 13-24A. The Mount Hope Bridge from Bristol to Portsmouth, R.I. The main span is 1,200 ft.; the side spans approximately 500 ft. The piers are solid except for a vertical hollow space 18 ft. square in the center of the lower portion. They are supported by spread footings, one being 38 ft. and the other 60 ft. below water. Both are founded upon sand and gravel with some clay. One anchorage is on rock; the other on wooden batter piles in a mixture of gravel, clay, and boulders. *(Designed by D. B. Steinman, Consulting Engineer, New York, N.Y.)*

FIG. 13-24B. The Ambassador Bridge over the Detroit River at Detroit, Mich. The main span is 1,850 ft. Notice the style of the anchorage and the use of light steel bents and towers to support the side spans. Consulting architect, Smith, Hinchman, and Grylls, Detroit, Mich.; designing engineer, McClintic-Marshall Co. (now Bethlehem Steel Co.). *(Courtesy of Bethlehem Steel Co.)*

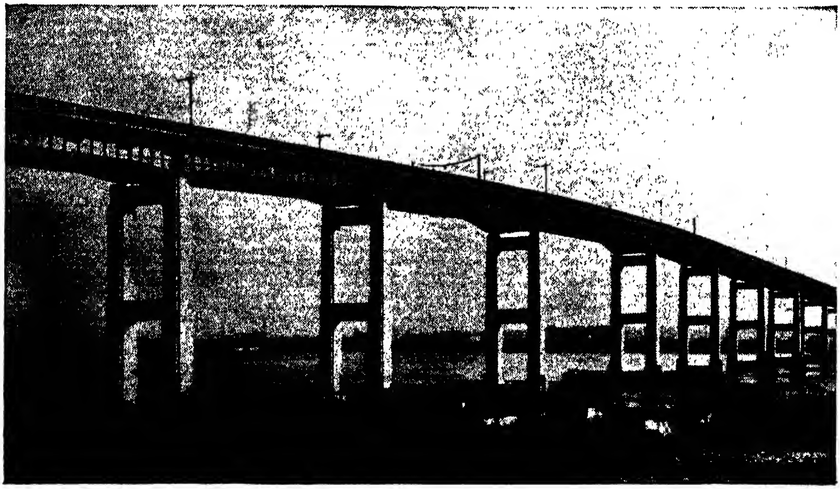


FIG. 13-25A. Thomas A. Edison Bridge over Raritan River at Perth Amboy, N.J. The central plate-girder span is 250 ft. Designing engineer, New Jersey State Highway Department; bridge engineer, Morris Goodkind. *(Courtesy of Bethlehem Steel Co., fabricators and erectors of the steelwork.)*

FIG. 13-25B. Piers of the north approach of the St. Georges Bridge at St. Georges, Del. The maximum height is 123 ft. above top of base. When piers are so tall, the bending moments in the shafts and portal caused by wind may be severe. The bending in a continuous footing or bottom strut would also be very large. For such conditions the designers used the idea of having a hidden member across the bottom that was designed to permit a small angular deformation, yet to limit the rotation and to offer appreciable restraint. *(Designed by Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y.)*

FIG. 13-26A. Construction of the Dravosburg Bridge in Allegheny County, Pa. This is a cantilever bridge. Part of an anchor arm is shown on falsework in the foreground. Notice the simple but neat cylindrical pier shafts. *(Courtesy of Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y. and the Department of Public Works, Allegheny County, Pa.)*

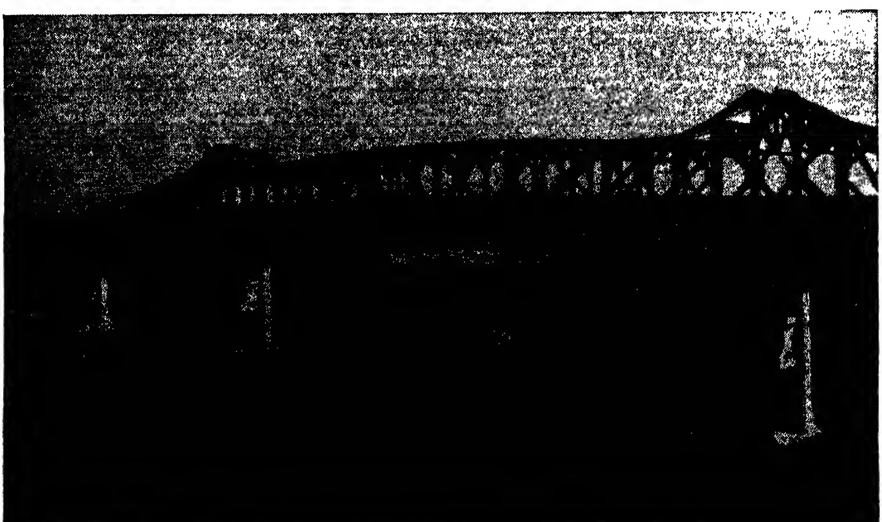


FIG. 13-26B. Bridge on the Pulaski Skyway between Newark and Jersey City, N.J. This is a cantilever bridge with a 550-ft. main span and 350-ft. anchor arms. Designing engineer, S. Johanneson, New Jersey State Highway Department. *(Courtesy of Bethlehem Steel Co., builders of the superstructure.)*

FIG. 13-27. The Peace Bridge over the Niagara River. The piers are founded upon rock. They are designed for a swift current having a normal velocity of 9 m.p.h. and a maximum of approximately 12 m.p.h. The normal depth of water near the center of the river is 22 ft.; the maximum about 27 ft. Notice the provision of ramps as icebreakers. The truss at the far end is over the canal. *(Designed by the Edward P. Lupfer Corporation, Buffalo, N.Y., Wm. Russell Davis, Consulting Engineer.)*

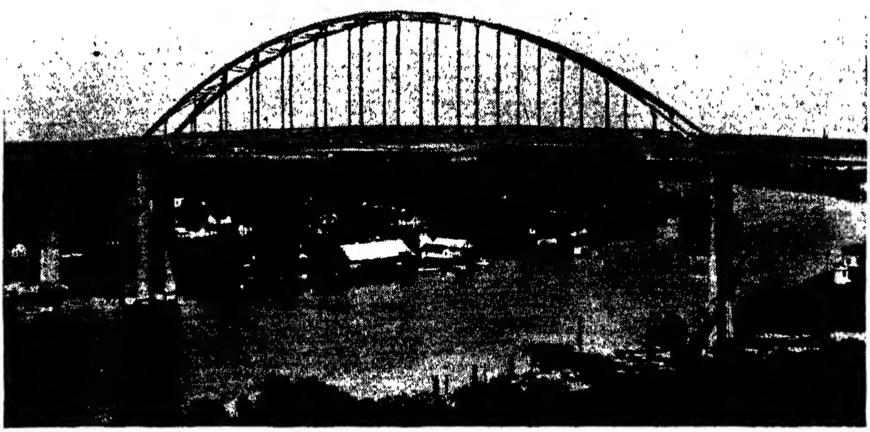


FIG. 13-28. The Chesapeake City Bridge at Chesapeake City, Md., over the Chesapeake and Delaware Canal. This is a 540-ft. tied arch. The piers are reinforced concrete, framed bents founded on H piles. The single-shaft approach piers are supported by cast-in-place concrete piles. *(Designed by Parsons, Brinckerhoff, Hall & Macdonald, Consulting Engineers, New York, N.Y.)*

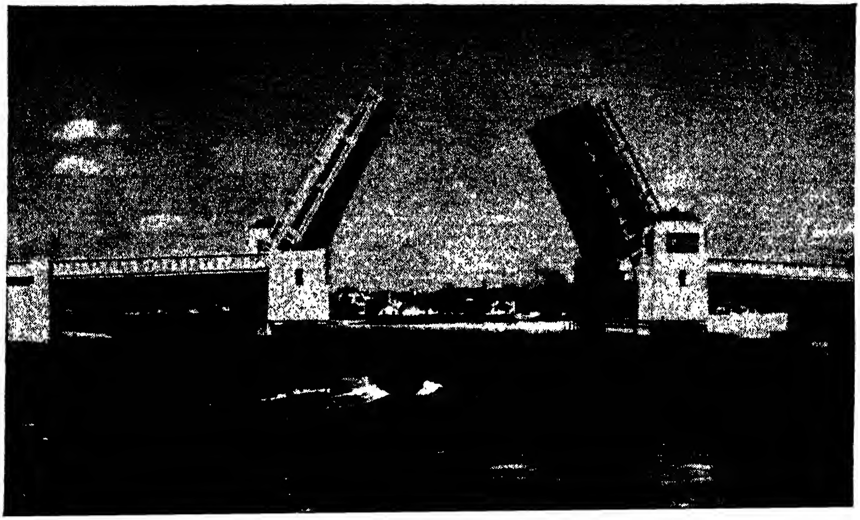


FIG. 13-29A. A bascule bridge across the Shark River Inlet, Belmar, N.J. The pier bases are approximately 22 by 69 ft., and are approximately 10 ft. below the river bed which consists of sand with small pockets of clay. The piers were constructed in steel sheet-pile cofferdams, and they are supported upon creosoted timber piles driven into hard packed sand. *(Courtesy of Howard, Needles, Tammen & Bergendoff, Consulting Engineers, New York City.)*

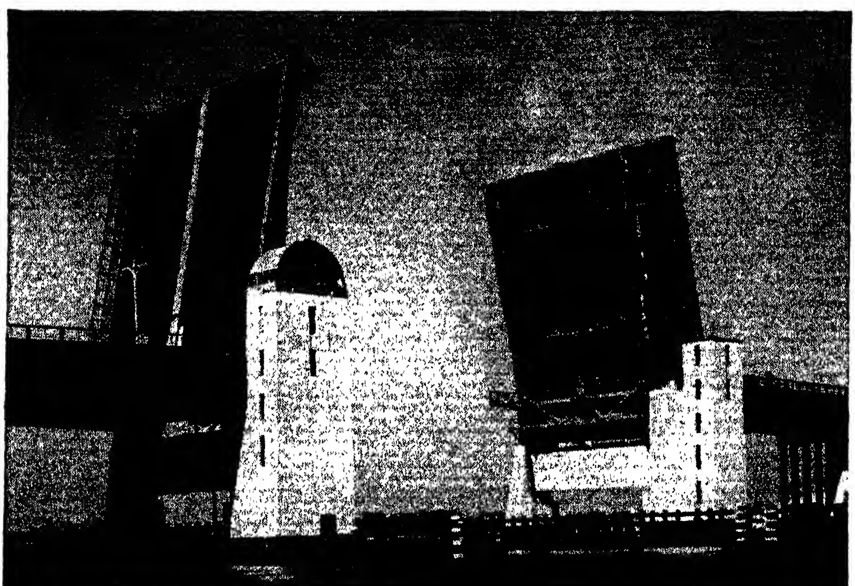


FIG. 13-29B. Mill Basin Bridge, Shore Parkway, Brooklyn, N.Y. The bascule span is 165 ft. Notice the types of pier under the approach spans. Designed by Waddell & Hardesty, New York, N.Y. *(Courtesy of Bethlehem Steel Co., builders of the superstructure.)*

FIG. 13-30. The Stickel Bridge on Route 25A over the Passaic River at Newark, N.J., built by the New Jersey State Highway Department. The piers of the lift bridge were built with pneumatic caissons to rock. One pier base is 60 ft. below mean water; the other, 40 ft. The foundations were designed by Morris Goodkind, bridge engineer, under Spencer Miller, Jr., Commissioner of the New Jersey State Highway Department. The superstructure was designed by Hardesty & Hanover, Consulting Engineers, New York City. *(Courtesy of Hardesty & Hanover.)*

FIG. 13-30A. Alameda Creek Bridge and overhead in California showing central circular piers. *(Courtesy of the California Division of Highways.)*

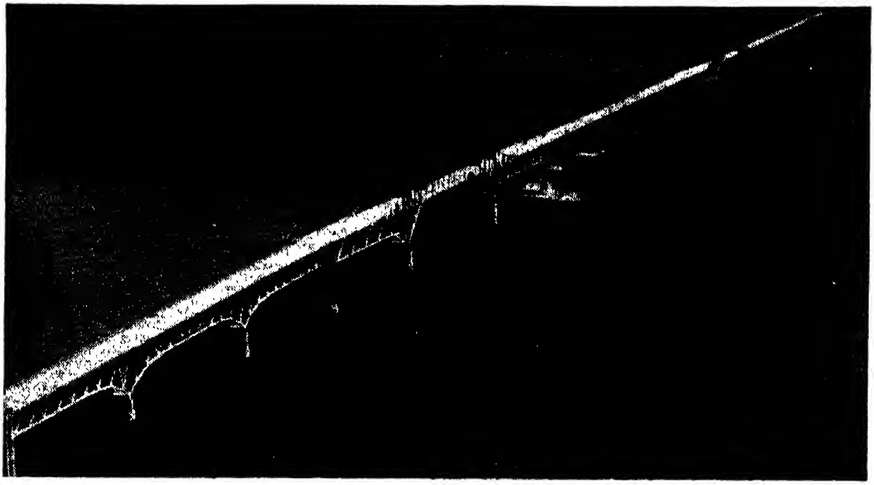


FIG. 13-30*B*. Bridge over the Susquehanna River at Havre de Grace, Md. A Wichert continuous span of 456 ft. with 332-ft. anchor arms and deck-truss approaches of Wichert continuous type. Designing engineer, J. E. Greiner Co., Baltimore, Md. *(Courtesy of the Bethlehem Steel Co., builders of the superstructure.)*

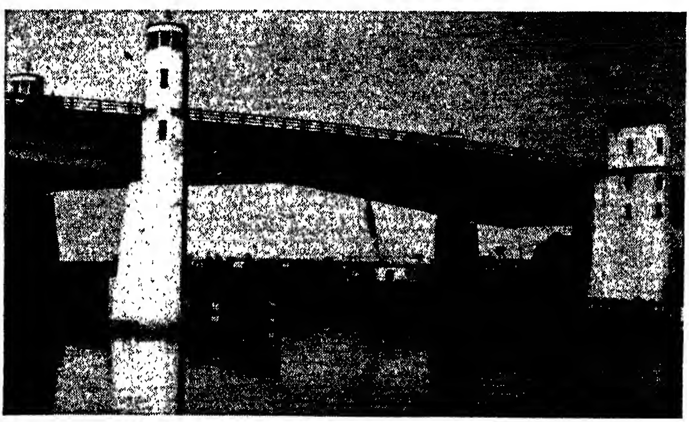


FIG. 13-31. The *Willets* Point Boulevard Bridge over the Flushing River. This is a bascule bridge on the Whitestone Parkway. It was designed for the Triborough Bridge Authority of New York by Hardesty & Hanover, Consulting Engineers, New York City. The piers at one end of the bascule bridge are on spread footings on sand; those at the other end are on timber piles. *(Courtesy of Hardesty & Hanover.)*

Many times there are special problems that call for special solutions. For example, Fig. 13-32 pictures a structure on 200 to 300 ft. of Jersey Meadows clay overlain by 8 or 10 ft. of peat. Lightness of piers was desirable, and so was an attractive appearance in keeping with the $90,000,000 project of which the viaduct is a part. The footings are generally continuous under four bearing pedestals constituting one pier, and they are sup-

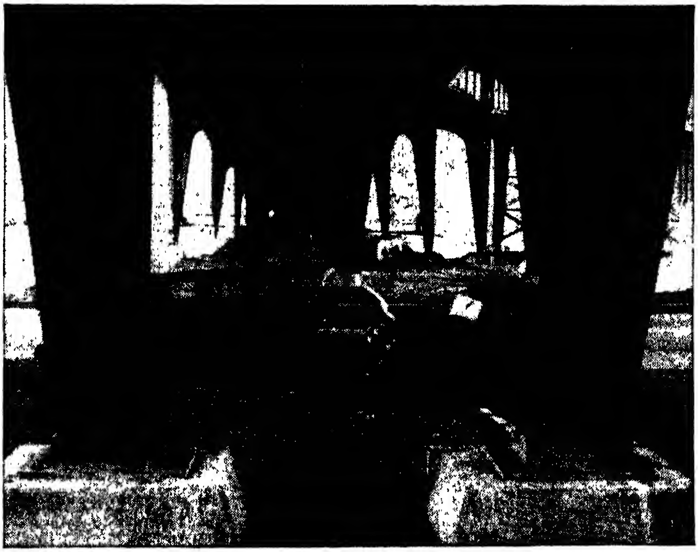


FIG. 13-32. Close-up of the supporting structure of the steel viaduct of the New Jersey approach to the Lincoln Tunnel at New York City. The tapered steel legs are terminated in interlocked, hinged bearings. The concrete pedestals rest upon continuous concrete footings that are supported by precast concrete piles. Horizontal forces are resisted by the frame action of the superstructure. *(Courtesy of the Port of New York Authority.)*

ported upon precast concrete piles. Longitudinal and transverse stiffness are provided by the rigid-frame action of the girders and tapered legs. Thus the bending is resisted by the superstructure whereas the shears are applied to the footings at the tops of the short pedestals only so that they do not cause large overturning moments on the pile foundations. The general arrangement of the structure is shown in Fig. 10-26.

At the left side of Fig. 13-28 is an approach pier in which simplicity may seem to have been carried to an extreme. However, the design is practical and economical. The simple shaft under the overhanging sides of the superstructure appears slender but not weak.

How best to support a pier of a desired type is another part of the problem. This will be illustrated more fully in Art. 13-10. It may be that these matters will occasionally react upon what is to be used for the visible portion of the pier.

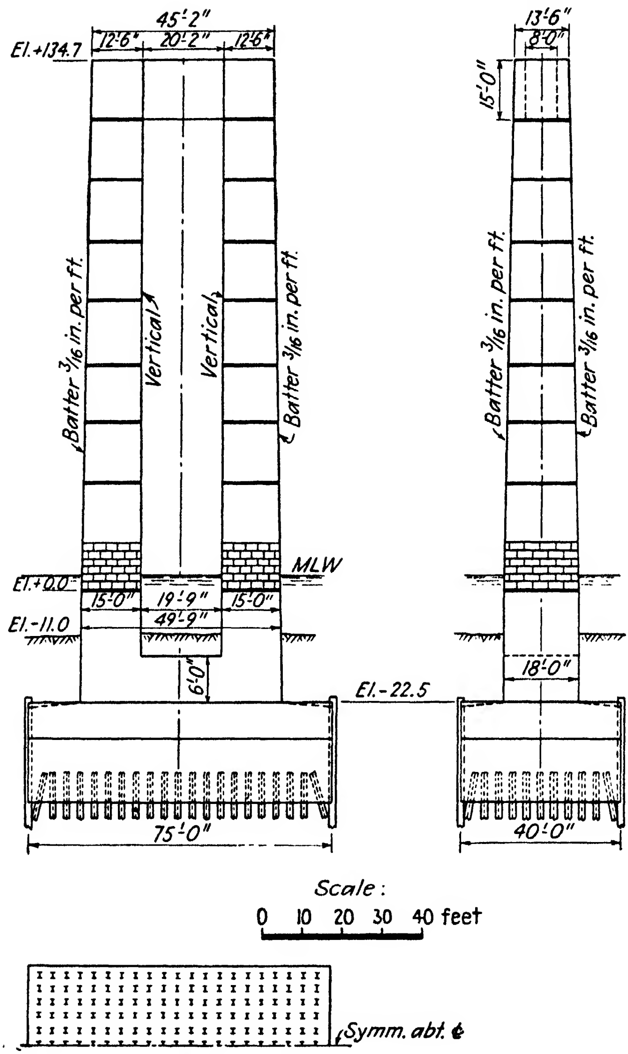


FIG. 13-33. The tall piers at the ends of the main span of the Chesapeake City Bridge, Chesapeake City, Md., shown in Fig. 13-28. (*Courtesy of Parsons, Brinckeroff, Hall & Macdonald, New York, N.Y.*)

13-8. Forces acting upon piers. It is difficult to set up general specifications for the forces to be applied to all bridge piers when testing their strength and stability. This is because of the differences in local conditions, the special characteristics of a particular structure, and the loads to which the bridge is to be subjected.

If other regulations are lacking, the following specifications may serve the purpose when one estimates the loads to be applied for the preliminary analysis of bridge piers and abutments. The diagrams referred to are intended to give a general scale of values; they cannot be exact because of the great variations in structures. In the descriptions, the word *transverse*

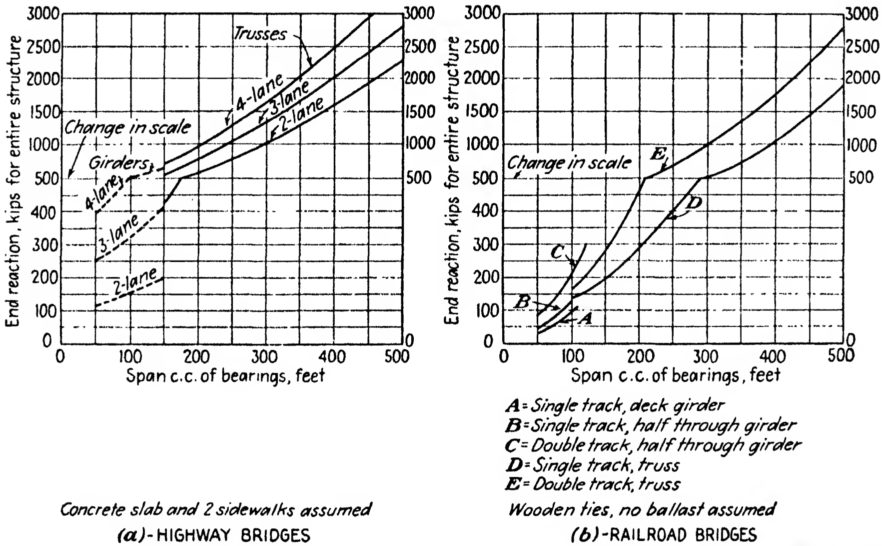


FIG. 13-34. Dead-load end reactions for highway and railroad bridges. (For preliminary estimates of loads on substructures only. Double the reactions to estimate total weight of bridge.)

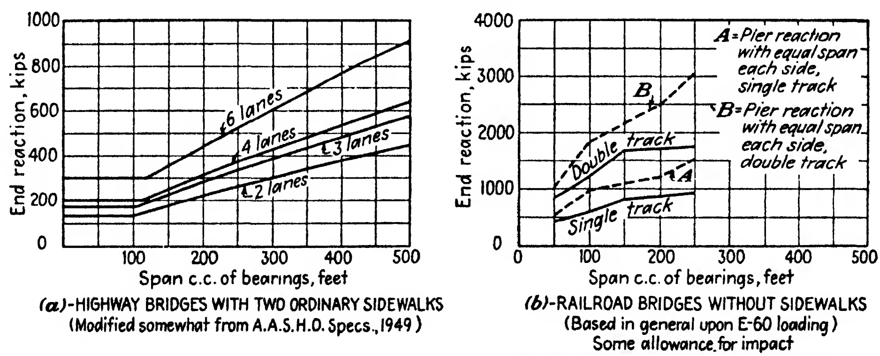


FIG. 13-35. Approximate total live-load reactions of bridges for preliminary analysis of substructures.

denotes a direction perpendicular to the longitudinal center line of the bridge whereas *longitudinal*, of course, means parallel to that center line.

1. Dead load. *a.* W_s, the estimated weight of the superstructure applied at the bearings. This should include the horizontal component of any unbalanced thrust caused by arch or rigid-frame action, including the resultant produced by the reactions of unequal spans or structures on adjacent sides of the pier. Figure 13-34 is for use in making preliminary estimates of the weights of some superstructures.

b. W_p, the dead load of the pier itself. This should allow for buoyancy of the submerged portion at 62.5 p.c.f. for fresh water and 64 p.c.f. for sea water. If the water level varies greatly, the resultant weight for both low water and flood conditions should be computed. Buoyancy should be considered in stability computations even for submerged foundations resting upon rock since water under pressure will probably penetrate seams or the junction of the concrete and the rock.

c. W_E, the weight of the soil above projecting footings. This should consider buoyancy for both low- and high-water conditions. It is useful when computing maximum bearing pressure under the pier, and when testing for overturning.

2. Live loads. *a.* R_{LL}, theoretical live-load reaction including no allowance for impact for highway bridges but with some allowance in the case of short railroad bridges. This should include the reactions, at the bearings, produced by the theoretical traffic loading condition that gives the critical load for the particular step being considered in the analysis. Just as for dead load, it should include any accompanying unbalanced horizontal components. Figure 13-35 is for use in making preliminary estimates of live-load forces. The impact allowance for railroad bridges is included in the short spans, then reduced arbitrarily to provide none for spans of 200 ft. or more.

b. R_{LO}, longitudinal force caused by live loads. Its position may be assumed to be at the bottom of the bearings. If applied at or above the roadway or rails, the force will cause a change of the reactions at the ends of the span plus a shear at the bearings. It is sufficient to include only the latter because the magnitude is uncertain. This force is caused by traction or braking forces and may be estimated from the data in Fig. 13-36. Of course, it must be consistent with the theoretical loading condition assumed for the simultaneous value of R_{LL}.

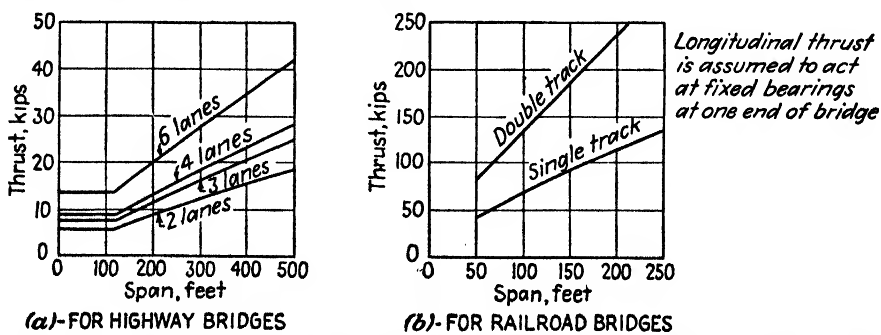


FIG. 13-36. Total longitudinal live-load thrusts for analysis of substructures. (Note: Transverse live load for highways = 0; for railroads, 20 kips per track on one track only.)

c. R_{LT}, transverse force caused by live loads. This will be neglected for highway bridges entirely. For straight railroad bridges, the lurching of a locomotive or cars will be assumed as 20 kips at the top of the pier, with allowance for one track only. Centrifugal force is to be included when a railroad track is on a curve and supported by superstructure.

3. Wind loads. *a.* H_{TS}, transverse force caused by wind blowing on the superstructure. The intensity of wind pressure, w_w, on a vertical surface may be assumed ordinarily as 30 p.s.f. but, in the case of some large structures and in areas where hurricanes occur, it may be advisable to increase

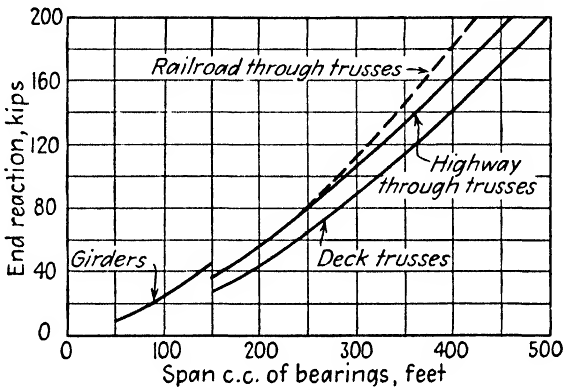


FIG. 13-37. Transverse normal wind reactions on highway and railroad bridge structures. (Note: For live loads, add 0.2 kip per lin. ft. for highways and 0.3 kip per lin. ft. for railroads.)

this 50 per cent. The total transverse force applied to the pier may be computed as the reaction produced by w_w acting upon an area equal to approximately 1½ times the estimated area of the side elevation of the spans on each side of the pier. When deep trusses are 30 ft. or more apart, it is safer to assume both trusses as completely exposed except at the floor system. Although this force H_{TS} should be applied at the computed center of gravity of the assumed side areas of the superstructure, its magnitude is so uncertain that it is usually sufficient to apply it at the bottom of the bearings except in the case of through trusses. Then it is satisfactory ordinarily to estimate the center of gravity by eye from a sketch of the side elevation of the structure. For these computations, all spans may be assumed to be simply supported even though this is not the fact. Some data for use in preliminary estimates of H_{TS} are given in Fig. 13-37. However, more careful estimates should be made for large structures.

b. H_{LS}, longitudinal force caused by wind blowing diagonally against the superstructure. This and a corresponding transverse force, H'_{TS}, will act simultaneously. The data in Fig. 13-38 are for use in making preliminary estimates of these forces. The minimum magnitude of H_{LS} is not zero, even with longitudinal wind. The direction of the wind that generally causes the worst condition at the pier may be assumed to be 45° from the normal to the bridge center line. This gives transverse and longitudinal forces that are each equal to $\frac{1}{2}H_{TS}$ except as affected by wind forces on exposed transverse members such as trusses. For the web members, it is assumed that these components are each approximately equal to the full wind on their side elevations.

In the case of tall piers and bents where overturning may be a hazard, it is desirable to increase the wind forces by 50 per cent, then to test the pier

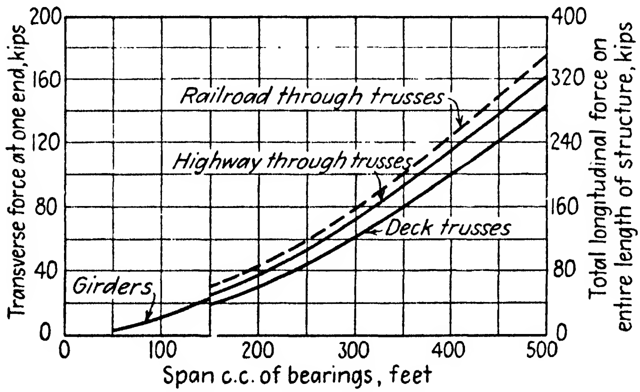


FIG. 13-38. **Transverse and longitudinal wind reactions for 45° wind on highway and railroad bridge structures. (*Note:* For live loads, add 0.1 kip per lin. ft. for highways and 0.15 kip per lin. ft. for railroads.)**

without live load on the structure. This is to allow for hurricanes. The safety factor against overturning in such a case may be reduced to 1.25 or thereabouts.

c. H_{TP}, transverse wind on the pier. This may be assumed to equal w_w times the estimated exposed area of the pier in side elevation at low water plus any secondary exposed area, as for example, the leeward column of a two-column pier. The resultant is to be applied at the estimated center of gravity of the forces concerned.

d. H_{LP}, longitudinal wind on the pier when the wind is blowing diagonally. This force and the corresponding transverse force, H'_{TP}, may be estimated on the basis of the total projected area of the pier normal to the wind direction times w_w. This force should then be resolved into the proper components parallel and perpendicular to the bridge.

4. Special forces. EARTHQUAKE FORCES. These are difficult to estimate and should be included in one's analysis only in regions where seismic shocks are probable. If the structure is on rock, the vertical forces from earthquake shocks may be neglected as far as piers and abutments are concerned, and the horizontal forces may be assumed to equal 10 per cent of the weight of the superstructure (omitting live loads) acting at its estimated center of gravity plus 10 per cent of the weight of the pier (with buoyancy considered) applied at the latter's center of gravity. The surrounding earth and water may usually be neglected since their action is so problematical and since they are present on all sides of the piers. If the substructure is supported upon deep soils, whether granular or cohesive, and whether piles are used or not, the shocks from earthquakes are likely to be more serious since the soil may magnify the vibrations somewhat as a dish of jelly which is shaken slightly. The earthquake problem is one deserving careful study. In this case, it is probably best to build a structure that is basically strong in its resistance to earthquakes.

Other forces. Wave action, the thrust from currents, and ice pressures should be included when they are important. These have been discussed previously. They should be assumed at their critical condition, acting at flood stage.

Collision. In navigable waters, boats might collide with piers during storms and foggy weather. This should be prevented, not so much for the sake of the pier as for the protection of the boats which are relatively like eggshells compared to massive foundations. The best protection is a fender of wooden piles a few feet on the channel side of the pier and curving around to protect both ends so as to deflect a ship channelward. The resilience of the wood renders it better for this use than steel or concrete. The current is generally moderate in navigable channels so that the fender will probably cause no serious scour or obstruction to flow.

Friction. When the expansion bearings under a superstructure do not function easily, large longitudinal forces may be set up at the top of a pier. Sliding steel bearings that are rusted badly may practically "freeze"; round rollers and segmental rockers may become so clogged and rusted that they cease to operate properly. The longitudinal forces caused by thermal expansion and by live-load deformations may then be very large unless the pier can tilt sufficiently or the superstructure can deform to accommodate the impressed deformation. Ordinary sliding bearings may cause frictional forces equal to 0.3 to 0.4 of the dead-load reaction as a minimum or 0.3 to 0.4 of the live-load and dead-load reaction as a maximum. The resistance of deep rocker bearings to movement is problematical, but it is usually assumed to be negligible. Obviously, therefore, the type of bearing to be used is important. When a pier is purposely anchored to the superstructure in order to steady its top, the pier should be sufficiently flexible to prevent harm to it and the superstructure as the latter deforms.

All the forces listed in the preceding paragraphs may make the design of piers seem very complicated. This is not really so. Some of the forces may not apply to a particular case, or they may be so small compared to others that they may be neglected safely. For example, the forces caused by wind and lateral live loads on a low massive pier in the rapids of a river may be tiny compared to those of ice, current, and the dead load. The next article will illustrate how these forces may be used.

In any case, good judgment is needed in deciding which forces are applicable and how they should be combined. For example, a continuous structure may be anchored to one pier or an abutment as in Figs. 13-19(*a*) and (*b*). Then the longitudinal wind and live-load forces on that entire unit of the superstructure must be resisted by this anchor pier or abutment as intended. If fixed at one abutment as well as at the intermediate piers, the stiffest and strongest unit will offer the most resistance, this probably being the abutment, as shown in Fig. 13-19(*c*). If, on the other hand, the structure is fixed at two piers as in (*d*), these should share the resistance and be able to hold the structure without dependence upon bumping at the abutment.

13-9. Stability computations. A pier must be strong enough in itself to hold up the superstructure without collapsing. This is seldom a serious problem because most piers that are reasonably proportioned are relatively massive so that they act as blocks rather than as slender columns. Naturally, the soils under a pier must also be able to resist the vertical and all other loads without tilting, sliding, or harmful settlement. This is where the difficulties usually arise in the planning of piers.

Of course, the bearing area of the soil at the base of a pier must support the dead load of the structure always. For design purposes, the vertical live loads should be included.

The inclusion of wind and other lateral loads with vertical live loads and dead loads introduces the question of the probability of these maximum combinations. The figures in Table 13-2 are believed to be suitable for the allowable maximum computed bearing pressure on the soil, in terms of the safe allowable bearing value p, when the theoretical loading conditions are combined.

TABLE 13-2. Allowable Pressures on Soils for Various Combinations of Loading

Dead load plus vertical live load (including impact when necessary)....	p
Dead load plus wind...	p
Dead load plus wind plus ice and waves........................	$1.5p$
Dead load plus live load plus wind............................	$1.33p$
Dead load plus live load plus horizontal live load..................	$1.33p$
Dead load plus live load plus wind plus horizontal live load..........	$1.5p$

The pressure from current is to be included with any of the preceding loads when it is important.

In the estimating of the long-term settlement of bridges on cohesive soils, the full live load and wind load are so problematical and temporary that their effects may be very small or negligible. In general, the dead load only will be considered in the case of bridge foundations. In such computations, wind, horizontal live loads, and all others of temporary duration will be neglected also.

The determination of the depth to which the substructure is to extend is a very important starting point that should be based upon careful explorations of the site, relative economy, safety, and the problems of construction. The allowable bearing pressure p should be selected as a starting point. This important decision is often difficult to make with any certainty of its accuracy. This matter has been discussed in Art. 4-8.

Practical problems are so varied that each must be studied by itself. The method of attack is illustrated by the following examples. One of the points to be remembered particularly is the fact that there are usually several combinations of loading to be investigated for each problem. It is assumed here that the general size and shape of each pier have been determined and that its safety is to be tested. The analyses are abbreviated somewhat in order to save space. The general specifications for loading set forth in the preceding article are to be used if no other data are given.

Example 1. Plan the general features of the base of proposed pier B of Fig. 13-39. Then test it for maximum soil pressure and stability. In general, the safety factor against overturning is to be 1.5 for the critical combination of loads. The soil profile and the assumed bearing values of the different strata are given in Sketch (c). The values for p are those of the superimposed unit load that may be added to the soils in excess of the pressures to which they are already subjected by the overburden.

This is a preliminary design to establish the general features of the structure. A check analysis is assumed to be necessary after the superstructure has been completely designed.

This pier is part of the approach to a high-level bridge across a river near the Atlantic Coast. A main highway passing at a skew between piers B and C necessitates a longer span than the typical 100-ft. ones if the piers are to be normal to the axis of the bridge, which is desirable for simplicity of the superstructure. The piers above ground are all to look alike. The steelwork from A to D is to be the same depth, and continuity is to be used in this three-span unit.

The superstructure is to be "anchored" to pier B. Therefore any longitudinal forces acting upon this 350-ft. section of superstructure must be resisted by this pier. The expansion bearings at A, C, and D are to be deep rockers that will cause negligible frictional resistance to longitudinal movement. Assume a wind load at 45° to the axis of the bridge.

The general type of pier and its top dimensions are shown in Figs. 13-39(b) and (c).

The weight of the pier above El. 215 is approximately as shown by W_P in Fig. 13-40(b). The estimated forces to be applied to the pier (as taken from the data in Art. 13-8) are given in the figure. These loads include longitudinal live loads and a 45° wind. In connection with them, notice the following:

1. The live load is estimated from Fig. 13-35(a) for two lanes as though the loaded length affecting the pier is 270 ft., because of the continuity.

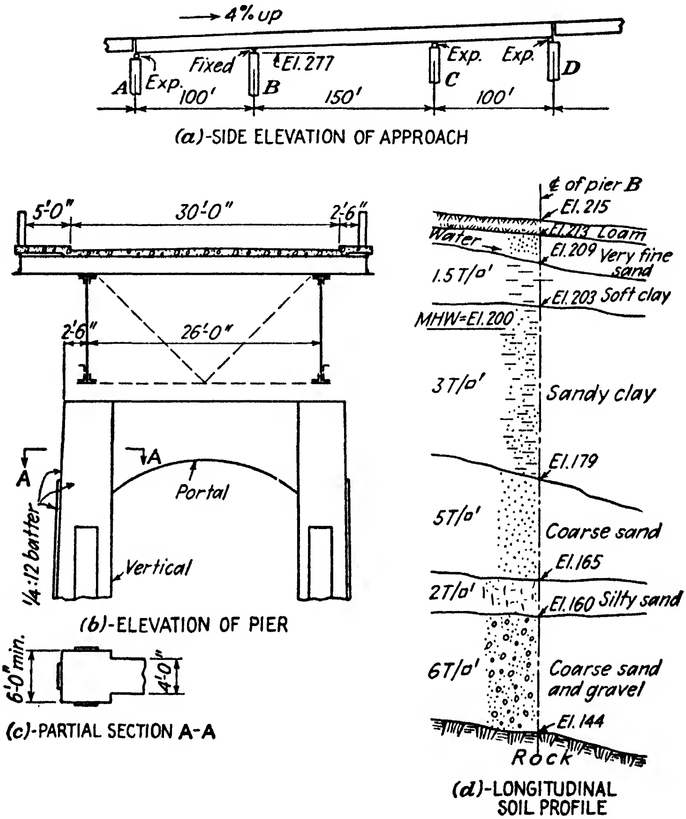


FIG. 13-39. Site and general arrangement for a pier in an approach to a large bridge.

This gives $R_{LL} = 280$ kips. This method of adding the adjacent span lengths (plus a little here) and considering it as a single span to get the reaction is used because of the concentrated loads applied at the reaction point in making the diagram. These need not be doubled.

2. The dead load of the superstructure is estimated from Fig. 13-34(a) for one span of 150 ft., plus one of 100 ft., plus a little for continuity, giving $W_S = 880$ kips.

3. The longitudinal live load R_{LO} is estimated from Fig. 13-36(a) for a 350-ft. span since all braking forces on the three spans must be resisted

by this pier. This gives $R_{LO} = 14$ kips. The probability of this load is questionable, but it is included for the sake of safety.

4. The wind load at 45° on the superstructure is estimated as follows from Fig. 13-38:

a. Transverse wind on one 150-ft. and one 100-ft. span is 25 kips for live load plus 75 kips for the structure. Then H_{TS} for 45° wind $= 100 \times 0.5 =$ 50 kips.

b. The longitudinal component H_{LS} on the 350-ft. structure is then approximately $H_{LS} = 50 \times 350/125 = 140$ kips. It is not correct to compute this as for a single 350-ft. bridge, the exposed area of which would be much larger.

5. The transverse wind forces on the pier are estimated as follows:

a. Assume two full end areas and compute their center of gravity.

b. Assume one full side elevation, as in Fig. 13-40(*a*), compute its area and center of gravity.

c. Add these areas to find ΣA, compute their combined center of gravity as shown on the drawings, and compute the total diagonal wind pressure on the pier as

$$H = \Sigma A \times 0.7 \times 30 = 40{,}000 \text{ lb., or } 40 \text{ kips}$$

Then each component will be

$$H_{TP} = 40 \times 0.7 = 28 \text{ kips}$$
$$H_{LP} = 40 \times 0.7 = 28 \text{ kips}$$

Now, how may the pier be supported when the soils are as shown in Fig. 13-39(*d*)? Here are some questions to think about:

1. Is the soft clay layer suitable to support spread footings under the pier? No. The structure is continuous, it is too heavy, the overturning forces are large, and better materials are near at hand below it.

2. Will the top layer of fine sand spread the loads over the soft clay? No. There is not enough sand to mean much after the footing is embedded sufficiently, and the clay is dangerous.

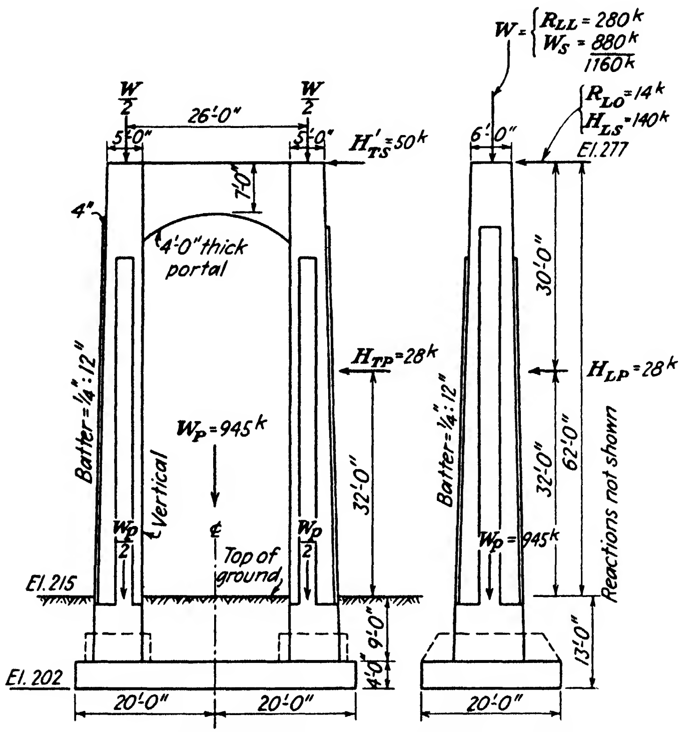
3. Should piles be driven down into the coarse sand? This seems to be a needless expense, and it is to be done only if nothing more practicable can be planned.

4. Is the 5-ft. stratum of silty sand dangerous? No. It is buried under a strong deep blanket of firm soils so that it cannot escape. It is so far down that the unit load on it will be small, and it is so thin that the total compaction will not be serious.

5. Disregarding the settlement problem, is it cheaper to use large footings at about 4 or 5 ft. below the surface and above the soft clay, or to extend the concrete columns down to footings on the sandy clay 12 or 14 ft. below the ground? The former will require nearly twice as much bearing area

as the latter needs. The deeper smaller footing seems preferable. Plan therefore to place the footing at El. 202, which seems to be below the soft clay for the necessary width.

6. Is it better to use two separate footings, one under each shaft, or a combined footing under both? Since this pier is an anchor pier for three spans of the approach, it should have good resistance against tilting longi-



(a)-SIDE ELEVATION (b)-END ELEVATION

FIG. 13-40. An anchor pier for bridge approach.

tudinally. It therefore seems best to have two footings with large width in this direction. The central portion of a wide combined footing would not be so effective as desired unless it were very thick, because of the application of the moments near its ends. Of course, differential settlement of the footings is to be avoided. Such settlement does not seem to be probable here. The portal will also help to hold the two columns or shafts together.

Therefore, try two footings 15 ft. wide, 20 ft. long (parallel to bridge), and 3 ft. thick. Place the bottoms on the sandy clay at El. 202. Allowing approximately 1 k.s.f. for the weight of the displaced soil, the maximum allowable bearing value for design, including all loads, will be

$$p = 1.5 \times 6 + 1 = 10 \text{ k.s.f.}$$

The approximate gross weight of the concrete and earth below El. 215 is 935 kips.
Then,

$$\Sigma W = 1{,}160 + 945 + 935 = 3{,}040 \text{ kips}$$

$$\text{Longitudinal } M \text{ above El. } 202 = 154 \times 75 + 28 \times 45 = 12{,}760 \text{ ft.-kips}$$

$$\text{Eccentricity at base} = e = \frac{12{,}760}{3{,}040} = 4.2 \text{ ft. (beyond middle third)}$$

$$\frac{30p'}{2} \times 3(10 - 4.2) = 3{,}040 \qquad \text{and} \qquad p' = 11.6 \text{ k.s.f. (too large)}$$

Next, try a combined footing 40 ft. wide, 20 ft. long, and 4 ft. thick as shown in Fig. 13-40. This is because much more area is obviously needed.

$$\text{Weight below El. } 215 = 1{,}260 \text{ kips (approx)}$$
$$\Sigma W = 1{,}160 + 945 + 1{,}260 = 3{,}365 \text{ kips}$$
$$e = \frac{12{,}760}{3{,}365} = 3.8 \text{ ft.}$$
$$\frac{40p'}{2} \times 3(10 - 3.8) = 3{,}365 \qquad \text{and} \qquad p' = 9.1 \text{ k.s.f.}$$

The moment transversely about El. 202, allowing one-third reduction because of the portal at the top only for separate footings, is

$$M_t = \tfrac{2}{3}\, 50 \times 75 + 28 \times 45 = 5{,}010 \text{ ft.-kips}$$

Assuming, as an approximation, that there is pressure over the whole base, the edge pressure from this moment is

$$p'' = \frac{Mc}{I} = \frac{5{,}010 \times 20}{\frac{1}{12}(20 \times 40^3)} = 0.9 \text{ k.s.f.}$$

The estimated maximum edge pressure then is

$$p = 9.1 + 0.9 = 10 \text{ k.s.f. (satisfactory)}$$

The safety factor against longitudinal overturning, using an estimated point of rotation 2 ft. inside the edge of the footing, is

$$\text{S.F.} = \frac{3{,}365 \times 8}{12{,}760} = 2.1 \text{ (approx)}$$

For no live load and a 50 per cent increase in the wind,

$$\text{S.F.}' = \frac{(880 + 945 + 1{,}260) \times 8}{1.5 \times (140 \times 75 + 28 \times 45)} = 1.4 \text{ (approx)}$$

This is a bit too low but the lever arm is problematical and the loading is an extreme case.

This plan will be accepted tentatively. After the superstructure has been designed and the loads are known, the pier should be checked. Because of the twisting required to develop the central area of this combined footing, it will be desirable to add the pedestals shown by the dotted lines in Fig. 13-40. The interior portion of the footing will then be designed as a beam spanning between these pedestals.

This problem shows that tall slender piers used as anchor piers may require more of a footing than one suspects at first.

Example 2. Assume that the pier shown in Fig. 13-41 is to be analyzed for bearing pressures and stability, for dead loads, live loads, and horizontal

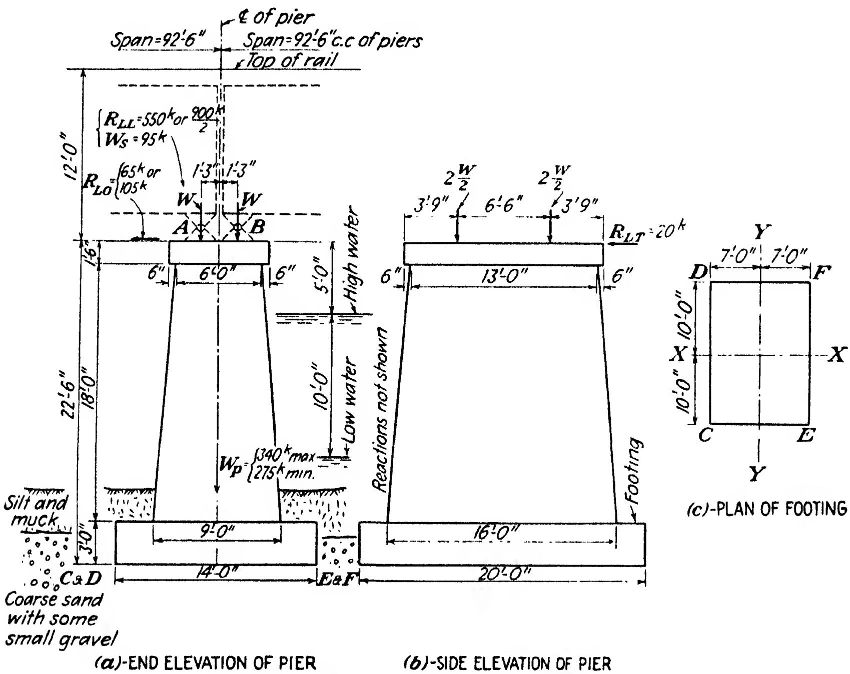


FIG. 13-41. A pier for a deck-girder railroad bridge.

traction or braking and lurching forces. This is a single-track railroad deck-girder bridge across a swampy unnavigable arm of a reservoir in northern New York. Current, waves, and ice are negligible. The ordinary allowable bearing value p of the sand and gravel is 4 tons per ft.^{2} For this load combination, according to Table 13-2, $p = 1.33 \times 8 = 10.6$ k.s.f. Is the proposed structure satisfactory?

For maximum pressures on the soil, the pier has the greatest effective weight at low water. The minimum effective weight at high water is critical for stability computations. The dead loads are relatively light, whereas the live loads are heavy. The smaller value of R_{LO} in Fig. 13-41 is for loads on one span only; the larger, for loads on both spans. For one span loaded, $R_{LL} = 550$ kips; for two, 900 kips.

The bridges have expansion bearings at their far ends and fixed bearings at this pier. Hence, all longitudinal loads on both spans are resisted by this pier.

The computations are as follows:

One span loaded; high water:

$$W = 275 + 2 \times 95 + 550 = 1{,}015 \text{ kips}$$
$$M_Y = 65 \times 22.5 + 550 \times 1.25 = 2{,}150 \text{ ft.-kips}$$
$$M_X = 20 \times 22.5 = 450 \text{ ft.-kips}$$
$$I_Y = \frac{20 \times 14^3}{12} = 4{,}600 \text{ ft.}^4$$
$$I_X = \frac{14 \times 20^3}{12} = 9{,}350 \text{ ft.}^4$$
$$A = 14 \times 20 = 280 \text{ ft.}^2$$
$$p_C = \frac{1{,}015}{280} + \frac{2{,}150 \times 7}{4{,}600} + \frac{450 \times 10}{9{,}300} = 7.4 \text{ k.s.f. (safe)}$$
$$p_F = 3.6 - 3.3 - 0.5 = -0.2 \text{ k.s.f.}$$

Neglect the slight inaccuracy of relying upon tension at this corner.

Stability safety factor for the longitudinal direction, using an assumed fulcrum 1 ft. inside the edge of the footing:

$$\text{S.F.} = \frac{(550 + 95)4.75 + 95 \times 7.25 + 275 \times 6}{65 \times 22.5} = 3.7 \text{ (safe)}$$

Two spans loaded; low water:

$$W = 340 + 2 \times 95 + 900 = 1{,}430 \text{ kips}$$
$$M_Y = 105 \times 22.5 = 2{,}360 \text{ ft.-kips (assume equal loads on bearings)}$$
$$p_C = \frac{1{,}430}{280} + \frac{2{,}360 \times 7}{4{,}600} + \frac{450 \times 10}{9{,}300} = 9.2 \text{ k.s.f. (safe)}$$
$$p_F = 5.1 - 3.6 - 0.5 = 1.0 \text{ k.s.f.}$$
$$\text{S.F.} = \frac{(900 + 190 + 340) \times 6}{2{,}360} = 3.6 \text{ (safe)}$$

13-10. Problems in planning a large pier. In the planning of the foundations of bridges, as stated previously, there are usually several possible and practicable designs that might be used in any particular case. It is important for an engineer to devise such alternate schemes; to compare their safety, cost, and practicability; then to make a wise choice among them. Of course, the selected plan should be consistent with the remainder of the bridge, both superstructure and foundations, so as to have all parts in harmony. The example in this article is given to illustrate such creative work.

Example. A highway bridge is proposed across the mouth of a stream in Connecticut along the north shore of Long Island Sound. One plan for the structure is shown in outline in Fig. 13-42(*a*). A map of the area is given in (*c*). The borings revealed the soil conditions shown in (*a*). Of course, the usual interpretations have been made in drawing the picture between borings. Plan the general features and dimensions of a suitable pier at point *B*.

The bridge is a cantilevered trussed structure with a tied-arch suspended span. The sheltered harbor is used extensively by small boats and pleasure craft. The length and clearance for the main span are based upon governmental requirements to accommodate the boats. The reaction at *G* will be large, and it seems that pinned but fixed bearings should be used here because expansion bearings would be difficult to make for such large loads. The pier then is an anchor for the portion of the superstructure between *H* and *J*, since the hanger at *J* permits longitudinal movement. Point *K* is made so as to prevent longitudinal swinging of the suspended span, and to transmit all longitudinal forces on portion *JK* to *K*. Pier *C* has been selected as the anchor for portion *JE*.

From the profile in Fig. 13-42(*a*), it appears that pier *B* is in what used to be the channel of an ancient stream that has been partly filled with glacial debris, clay, and silt. The water is deep, and the rock is nearly 90 ft. below mean high water. The tide range is only 5 ft., and the tidal currents are small. There is seldom serious icing in the Sound.

What types of pier are suitable here? How does one prepare an answer to this question? Of course, such a problem requires careful and detailed study. The present purpose is to devise general designs, then to judge their probable safety, economy, and practicability. The engineering staff can then prepare estimates and general drawings of the tentatively adopted design for inclusion in a report to the state highway commissioner.

The first studies might be made by freehand sketches drawn on cross-section paper so that they are reasonably to scale. However, such studies will be shown here by the drawings in Figs. 13-43 to 13-48.

The following paragraphs refer to the correspondingly numbered illustrations:

Figure 13-42(b). This shows rough estimates of the forces that may be applied at the top of pier *B*. They are sufficiently large to indicate at once that reliance upon the clay stratum is inadvisable; the silt is even worse. Therefore, the loads should be transmitted at least to the sand and gravel, 100 ft. below the bridge bearings.

Figure 13-43. This assumes that two circular steel open caissons are to be sunk to rock, filled with tremie concrete, and capped by a concrete block on which the two-column-and-portal pier is to be placed.

1. These caissons may be handled from a temporary platform on piles that are driven around the site of the pier. They are practically large steel pipes that are to be built in sections and welded together at the site. They

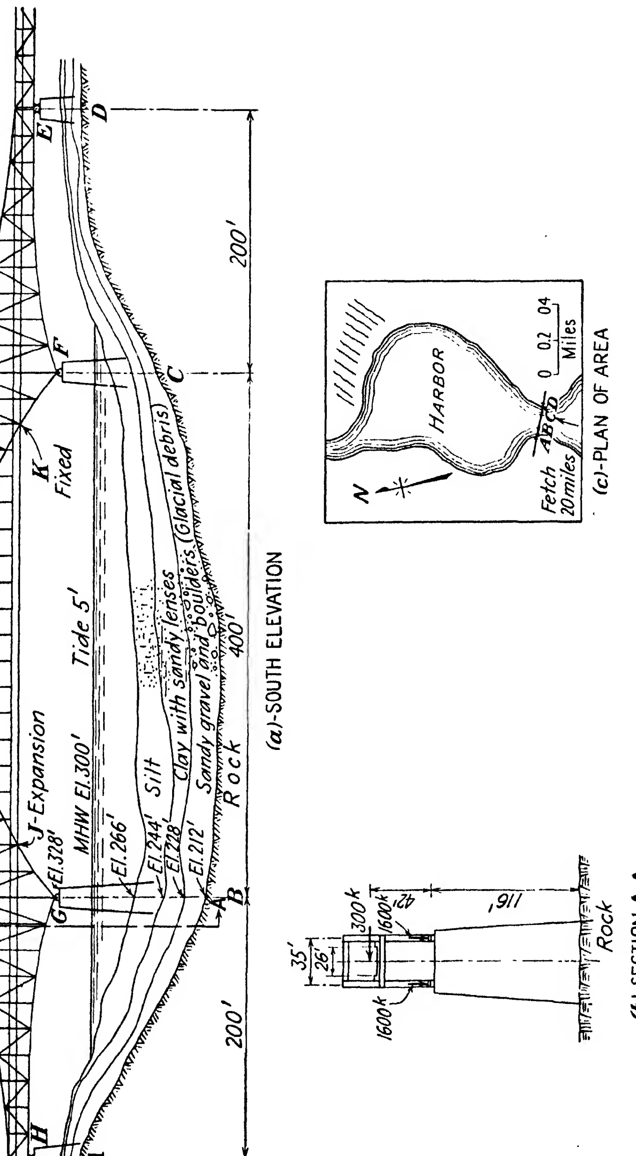


FIG. 13-42. General arrangement of a large cantilever bridge with a tied arch as a suspended span.

must be supported from the platform until they are long enough to rest in the silt without submergence of their tops. A sufficiently long piece of caisson might be built in a horizontal position on the platform, then lowered by derrick to rest upon the soil. This work would not be difficult.

2. These relatively narrow caissons will be difficult to steer when sinking by open excavation under water. They are too large to be driven. Jetting may assist in overcoming skin friction. Jacking against the platform or heavy weights on it will probably be needed to produce sufficient downward

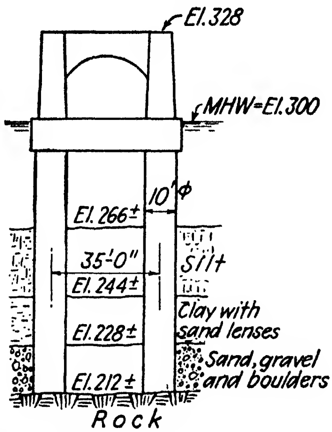


FIG. 13-43. Pier with two circular steel caissons filled with concrete.

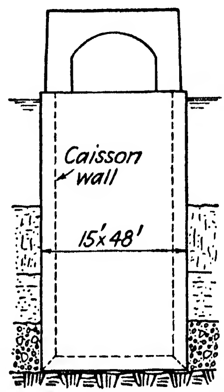


FIG. 13-44. Concrete-filled caisson.

thrust. Until well into the soil, these caissons may be disturbed by a severe storm.

3. These small caissons are likely to get hung up on the boulders above the rock. If so, what can be done? Perhaps these boulders can be displaced by digging or careful blasting; if not, the situation will be serious. The magnitude of these difficulties is uncertain, but such hazards should be avoided if possible.

4. This design may be all right for vertical loads, but its lateral stability does not seem very good. The silt will have some steadying value; the soils below it can be relied upon for lateral support, but they grip only the lower portion of the pier so that they are not so effective as one would wish. Reinforcement in the caissons and cap will help by providing some frame action. The caisson shells should not be assumed to provide strength since the upper portion will probably rust off eventually. These caissons are even less effective in resisting longitudinal forces because they are so narrow.

5. Thorough cleaning of the bottom is necessary to ensure good bearing of the concrete on such small areas. This can be done reasonably well by clamshell buckets and jets if the boulders do not interfere.

6. The columns and portal above the cap seem practicable.

Figure 13-44. This pictures a rectangular open caisson 15 by 48 ft. sunk to the rock, and having a two-column-and-portal structure on top of it.

1. Would the larger size eliminate the possibility of the caisson's stopping on a large boulder under the cutting edge? This danger is not so great as with the small caissons even though the cutting edge is longer. This is because the greater area provides more opportunity to get at and displace the obstructions by underwater excavation. If a pneumatic caisson is used, the work can be done safely because the men can get at the obstructions directly and prepare the bottom to receive the concrete. However, compressed-air work is very costly and should be used only when necessary or advisable. Here the depth is within the range for such work.

2. Such a caisson might be made of steel or concrete. It might be floated into place like a barge, then anchored, built up, and sunk gradually. This may be difficult work and dangerous in case of a severe gale during construction. The caisson will probably have to be nearly 50 or 60 ft. high before the resistance of the silt against the cutting edge will support it firmly. Hence the structure will be very "top-heavy."

3. Could the caisson be sunk more safely by the use of the "sand-island" method? It could. However, in such deep water and on so much silt, the enclosing sheet-pile structure would be very expensive, but it would probably be safer and the caisson could be steered more accurately. This scheme, although costly, will assure safety. The caisson might be sunk initially as an open caisson, then converted to a pneumatic one by adding an airtight roof if the boulders cause serious trouble.

4. This caisson requires more concrete, or concrete and steel, than would the small ones. However, the larger base and greater weight are likely to make it much safer. This may be worth the cost, considering how small a part it is of the cost of the entire bridge.

5. For economy and to avoid trouble from boulders, could the caisson be sunk to the top of the gravel and supported directly upon it? The area of the base is so large that this may be feasible. Even though the distance to rock is not great, that extra distance may be troublesome to excavate. Scour is not a hazard here. If the gravel is found to be dense, this idea seems attractive. However, it might be advisable to increase the width of the caisson to 16 or 18 ft.

Figure 13-45. This plan is based upon a sort of cofferdam and open excavation underwater. A structural-steel frame is to be supported by several steel H piles driven to the boulders or rock. Outside of this frame is to be driven a single row of heavy Z-shaped steel sheet piling. The silt, clay, and other soils are to be excavated by clamshell buckets clear down to the rock. When this is cleaned off, a solid rectangular tremie-concrete shaft is to be built up to about the water line. This is the foundation for the visible pier. The piling and steelwork are then to be removed, and backfill will be dumped around the pier.

1. The boulders can be removed because the area is big enough to enable one to get at them. The total excavation is greater, but the work is not costly.

2. The cofferdam material is recoverable. Since the water remains on the inside, the inward pressures on the sheet piling are not too serious. However, several sets of bracing will be necessary. Sheet piling over 75 ft. long is difficult to handle and drive. Therefore, these or shorter lengths are to be used and driven by a follower, making a submerged cofferdam, unless

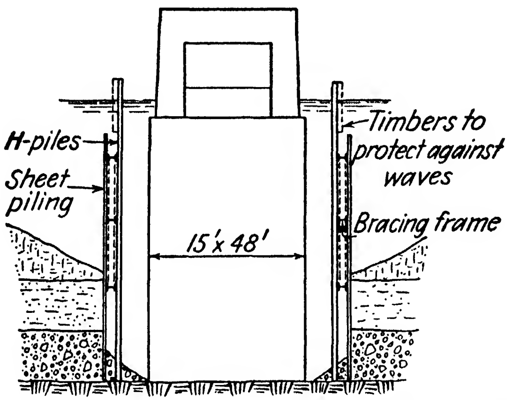


FIG. 13-45. Tremie-concrete pier shaft and submerged cofferdam.

special equipment is available to handle long lengths that will project above water.

3. How can the bracing of the sheet piling be erected? A considerable amount of the silt might be removed from the area first by hydraulic or open dredging, creating a big hollow in which to lower the steelwork. A bolted frame can be made with several sets of wales and having posts and diagonals on all four sides to form trusses. This frame can be assembled and lowered from barges to the bottom or held by temporary piling. The H piles can pass through prepared yokes or holes and be driven down. The frame may then be suspended from the tops of the piles. The H piles can also be used as the supports for a working platform. All this will require considerable troublesome work.

4. Is the form for the tremie concrete expensive? It would seem that this is a place where it would be economical to build a set of steel forms about 12 ft. high. These might be so designed that they could be lowered to the bottom and filled with tremie concrete. Later the forms could be broken away by ratchet-operated screws, wedges, or some such device. The forms could be raised, greased, lowered again, and tightened up against

the top 2 ft. of the previous pour. The concreting can thus proceed in lifts of about 10 ft. The area of the shaft should be large because tremie concrete is variable, and there may be some laitance that cannot be washed off by a jet of water. If such a scheme is adopted, the forms can be used again for pier *C*. At least, the elimination of complete forms for the piers is worthy of study. If full forming must be used, light forms with cross ties are possible, and they probably will not be salvageable.

5. Would this pier be stable? It probably would be if sand or gravel backfill were placed around it evenly and carefully. This might extend up near the top of the silt.

6. Could open excavation be used without the cofferdam, the pier itself being as outlined here and the work being done from barges? It might, but the "runny" silt would have to be removed from a large area. Storms and tidal currents might continually cause silt to be deposited in the excavated area. However, the idea is worthy of careful study.

7. Could the cofferdam be driven to rock and made just the size of the pier so that it could become the forms for the tremie concrete and be left permanently in place? It might. However, the boulders at the bottom might cause serious difficulties, as with a caisson; bracing should be installed near the bottom of the sheet piling to resist the inward pressures. The steelwork would not be recoverable. This last idea does not seem to be attractive, but estimates of total cost may show that it is.

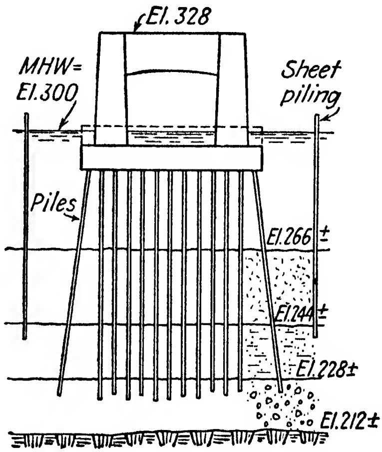


FIG. 13-46. Short pier on long piles.

Figure 13-46. This sketch shows a relatively short concrete pier on a spread footing supported by vertical and batter piles. The footing may be 8 or 10 ft. below low water. The sheet piling is to be on three sides, or possibly all around the area in order to protect against waves. It should have a fender to keep boats from hitting the projecting footing, or the base might be deep enough to bring it above high water.

1. This foundation will be relatively springy laterally in spite of the batter piles.

2. Wooden piles could be cut off easily, but they are not good as columns for such service, and they are subject to attack by marine borers. If they deteriorated, nothing could be done about it except at great cost.

3. Steel H piles could be driven to the boulders or rock and would be good columns. However, their resistance to corrosion from circulating sea water raises serious doubts regarding long-term service.

4. Steel pipe piles filled with concrete could be used. However, it would probably be difficult to unwater them, and tremie concrete in such small members is likely to have weak spots that endanger the pile after the steel has rusted away. If pipes 24 in. in diameter are used around steel H piles and the space filled with concrete to encase the piles, this concrete may disintegrate after the thin pipes have corroded away. With any piles, the boulders are likely to cause serious trouble, but steel H piles are the toughest for such driving conditions.

5. Precast concrete piles would be very long and hard to handle. Driving into the boulders might damage them. The exposure of such slender members to disintegrating elements in the sea water is still hazardous. Any type of cast-in-place pile is probably still less desirable.

6. Cutting off and capping the piles is a troublesome job. So is forming the bottom of the concrete cap when built in open water, provided the ends of the piles project into the cap and are gripped as they should be.

7. The vertical and lateral loads are so large that such stilted construction as this does not seem advisable. To lighten the load somewhat, a steel rocker bent might be used instead of heavy concrete. However, it is not desirable to have such steelwork subjected to wave action and salt spray, and the superstructure would have to be anchored at the outer ends A and D, which is not preferable.

Figure 13-47. This scheme is an attempt to make a compromise arrangement. A cofferdam is to be used so that most of the silt can be removed. Short steel H piles are to be driven to refusal by means of a long follower. Some gravel is to be dumped around the piles to consolidate the silt and hold the concrete. A short wooden form is then to be slid down inside the cofferdam, and a thick tremie-concrete footing is to embed the tops of the piles. A heavy tremie-concrete shaft is to be built nearly up to low water line, then capped with reinforced concrete. The pier shafts are to be placed on top of this cap.

1. This scheme avoids deep excavation, and the cofferdam can be shallower and lighter than that in Fig. 13-45.

2. The load is ultimately carried by piles to the rock or boulders. Many piles will be required, and some questions may still be raised as to their reliability unless the assumed design load per pile is conservative. This is largely because of the boulders.

3. Longitudinal loads cause the structure to tend to rotate on the piles. The pressures on the piles near one edge may be severe. This should be examined further. A gravel backfill up to the top of the silt would offer

considerable lateral support. Of course, the piles must hold the load of the backfill above the footing.

4. If piles are to be used at all, this scheme is safer than that of Fig. 13-46. However, it is heavier, requires more piles, and is more costly. It seems to be almost down to a good foundation but not quite so.

Figure 13-48. Here is a pier to be supported by a group of pipe caissons 6 ft. in diameter. They constitute "overgrown" piles.

1. Probably the boulders would cause even more serious trouble with these than with the 10-ft. ones of Fig. 13-43. Even by use of rotary sinking methods, satisfactory results are questionable.

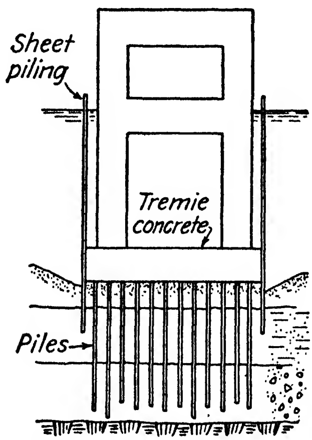


FIG. 13-47. Pier on short piles.

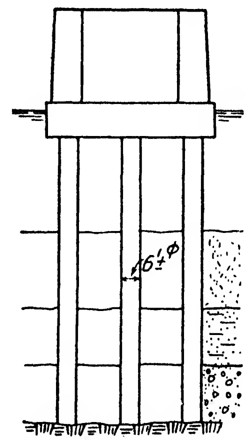


FIG. 13-48. Pier on six small caissons.

2. These slender members do not seem to be appropriate for a bridge like this. The strength of the foundation should be completely reliable.

Conclusions. Is this discussion of Figs. 13-43 to 13-48 confusing? By no means have all the possibilities been explored. However, the reader has been shown a little about the planning and comparison of such foundations. An engineer should not accept any scheme until he has explored whatever practicable alternates he can devise.

In this case, since the safety of a very important structure is involved and since it is a structure that is to endure for many years, it seems that a plan based upon the use of the caisson of Fig. 13-44 or the open-excavation scheme of Fig. 13-45 would be advisable. This is partly because almost any eventualities that may develop when the bouldery stratum is reached can be overcome, a strong and massive base can be built upon rock, the shaft can be steadied firmly by the deep soil or backfill around it, and there will be no question about its resistance to any and all anticipated forces. One advantage of the cofferdam scheme is that the cofferdam and all construction accessories can be used first at pier *B*, then they and the same methods can be used again for the building of the shallower but similar pier at C.

It is now desirable to plan each of these schemes of Figs. 13-44 and 13-45 in sufficient detail to make fairly good estimates of cost and studies of the construction problems. If the caisson can be supported upon the gravel, so can the pier built inside the cofferdam, and the latter can very easily have a wide base to provide longitudinal stability.

Without the benefit of estimates, the author is inclined to favor the cofferdam scheme of Fig. 13-45, provided the pier is supported upon firm gravel and the base is widened. This is partly because the cofferdam and forms seemingly can be reused for pier C, thereby reducing the total cost whereas with the caisson scheme two separate caissons will be needed unless a different scheme is used for pier C.

13-11. Bearing details. Knowledge of the dimensions and details of the bearings supporting the superstructure is important in the planning of a bridge pier. The accompanying drawings of some bridge bearings are given for the purpose of suggesting types and possible details. The proportions can be established when the loads and details of the superstructure are known.

The following are a few general comments regarding bridge bearings and pier tops:

1. Welded or cast metallic bearings for steel bridges

1. Except for small spans with light loads, special care should be used to see that the centralization of the load is maintained even when the superstructure deflects under the action of live loads. In other words, the pressure should not "ride one edge" of the bearing.

2. Pins in steel bearings centralize the load in general. However, angular movements or tendencies of the end of the superstructure to rotate slightly may cause wear on the pin and pin supports, if rotational motion occurs, a large restraining moment if friction keeps the parts from rotating, or some combination of the two. The bearing in Fig. 13-50(f) is especially likely to wear the pin and pin holes. Deflections from live loads as well as thermal changes will cause small movements repeatedly.

3. Pins fully supported in half-round grooves in the bearings for practically their full length are generally better than those having large shearing stresses and high bearing pressures on local pin plates. Compare Figs. 13-49(g) and (h).

4. Single rockers permit practical centralization of bearing pressures and expansion movement at the same time. However, a single-line (or long narrow area) bearing cannot be used for extremely large loads without excessive length. Figure 13-50(i) shows a twin-rocker arrangement.

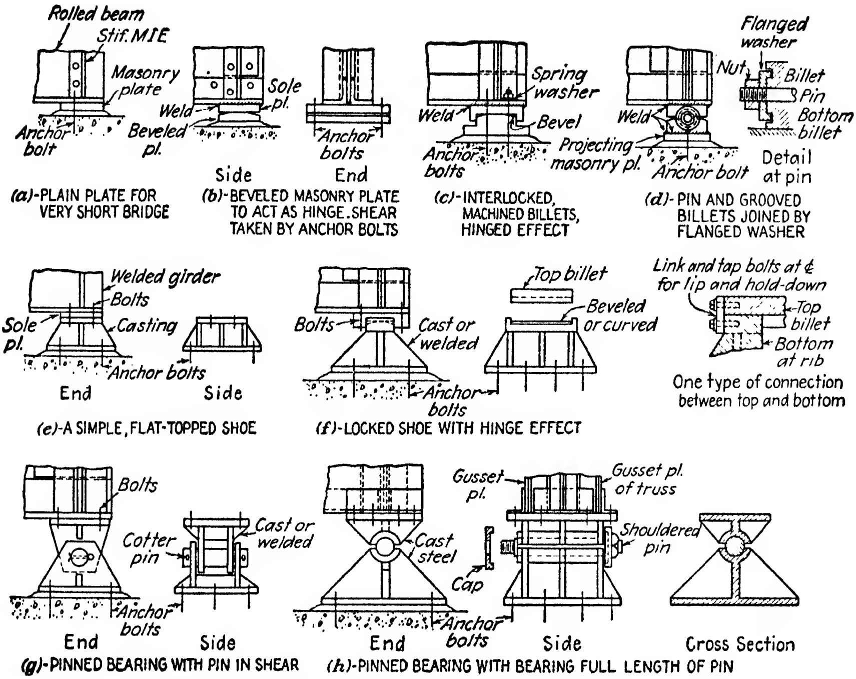


FIG. 13-49. Some details of fixed bearings for steel bridges.

5. Roller nests under heavy expansion bearings are useful in order to spread the load over a large area of concrete through several line bearings. Small cylindrical rollers have a low bearing value per inch of length and are likely to be difficult to maintain properly. Large circular rollers make the bearings unduly large. Therefore, segmental rocker nests, as in Fig. 13-50(*k*), are preferable.

6. With large single rockers, dowels or some other means should be used to prevent creep or twist of the rockers, and to resist transverse shearing forces. Friction alone may resist the latter, but some mechanical device is preferable. Bar guides are desirable to hold rocker nests in the right alignment, and side links are needed to force all rockers to maintain correct relative position.

7. Anchor bolts are generally used in masonry plates of expansion bearings. They may also be used to tie the structure down mechanically, as in Figs. 13-50(*a*) and (*b*). Shouldered bolts prevent pinching when the nuts are tightened. Flanged caps like that in Fig. 13-49(*d*) will hold the parts together. The anchor arm of a cantilever bridge may be tied down strongly, as in Fig. 13-50(*h*). This holding down is important where floods or gales may possibly cause uplift. A short heavy viaduct superstructure does not need such an attachment.

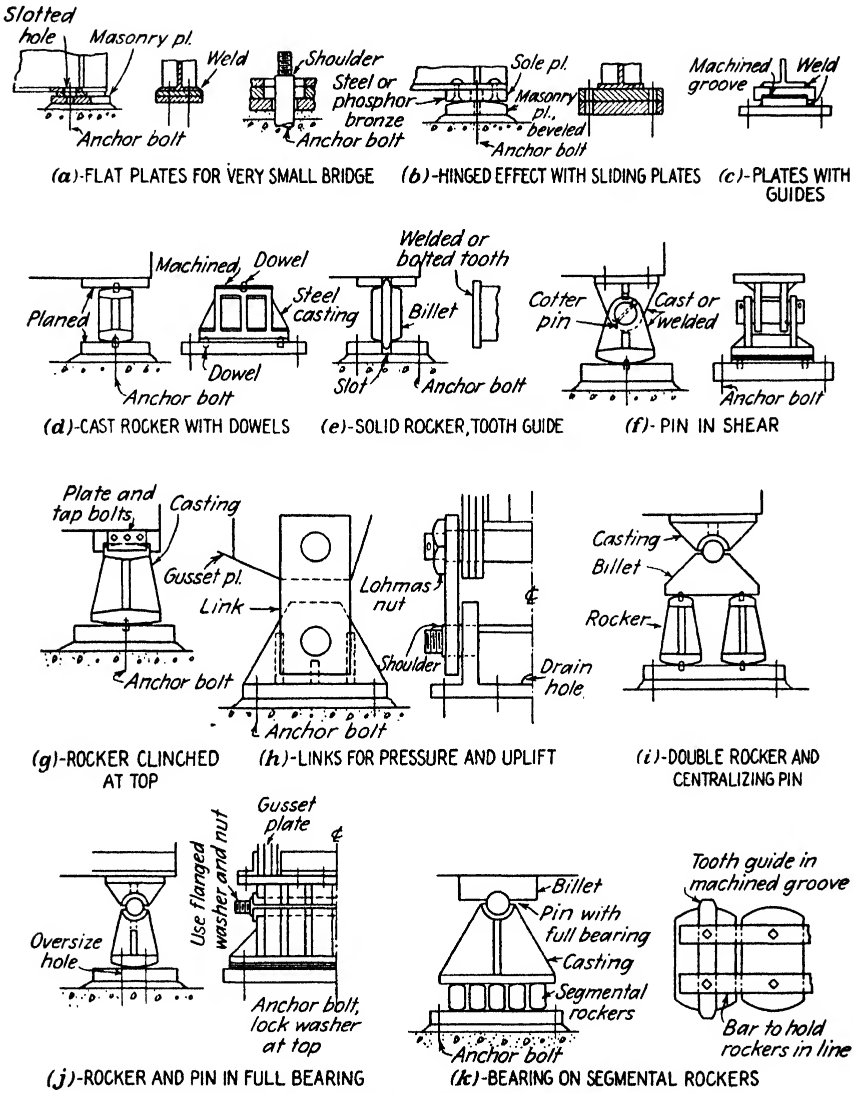


FIG. 13-50. Some details of expansion bearings for steel bridges.

8. Bearings should be accessible for proper maintenance.

9. Fixed bearings should be fastened to the concrete by anchor bolts sufficiently to ensure the prevention of sliding and tipping under the action of any forces that are expected to be applied to the superstructure. Anchor bolts should be of large size to allow for some corrosion.

10. Phosphor bronze or some other nonferrous material is desirable for sliding-plate bearings because of lower friction and prevention of "freezing"

by corrosion. Sliding bearings should be used only for light structures because they do not usually move easily. They should also be limited to use with short spans of 50 ft. or less in order to avoid the tendency to cause large edge pressures.

11. Bearings should be sturdy and strong, with metal thick enough to avoid weakness in case of considerable eventual corrosion. Cast steel,

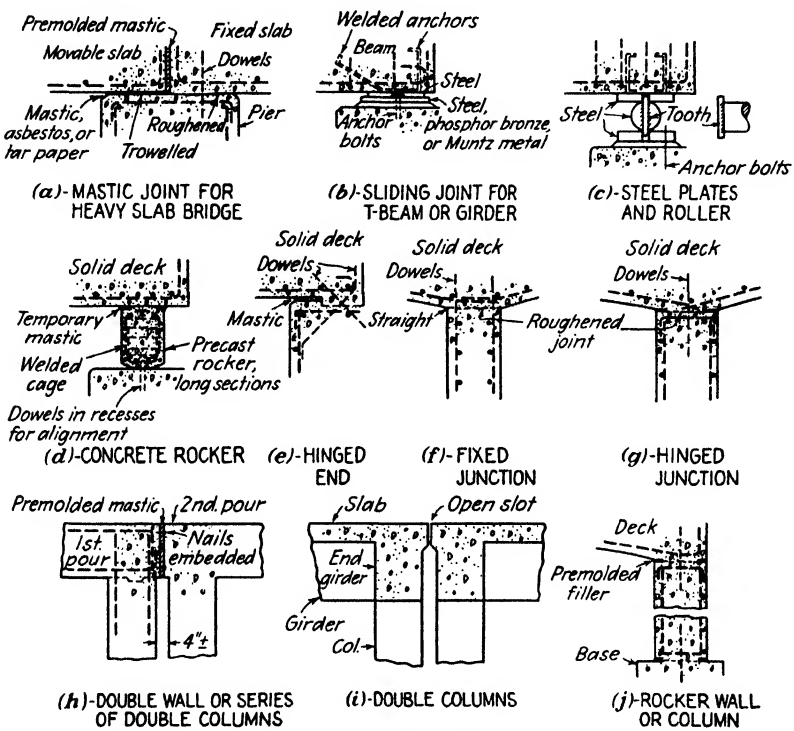


FIG. 13-51. Some details of bearings for reinforced-concrete superstructures.

riveted structural steel, or welded structural steel are best; cast iron is for use in very light structures only.

12. Bearing plates over 2 in. thick should be planed to secure even bearing of steel on steel, unless the shop can guarantee proper bearing without this. Bearing surfaces of castings should be planed also. Similarly, the proper finishing of rockers, rollers, and pins is essential. Steel plates or castings bearing on masonry need not be planed if the bearing is grouted.

2. Bearings of concrete structures

1. Sliding bearings under heavy concrete beams or flat slabs are not very satisfactory, not even with short spans. If used, it is desirable to have nonferrous bearing plates, one anchored to the pier and the other to the

superstructure. Concrete will not slide well on other concrete, not even when an asphalt layer separates them, because the asphalt may harden or be squeezed out.

2. Special care should be used in planning the reinforcement at fixed as well as at sliding bearings in order to prevent cracking locally because of the tension required to move the sliding bearing when the superstructure shortens.

3. Metallic rollers or rockers may be installed at expansion bearings if sufficiently strong bearing plates are used to spread the localized load over a suitable area of concrete. Precast concrete rockers, as in Fig. 13-51(*d*), may be used if the loads are not too heavy.

4. Flexible double columns or walls are often preferable at an expansion joint in a series of heavy concrete spans.

5. When an intermediate or end bearing is not reinforced to form a rigid joint, high edge pressure caused by the deflection of the superstructure may be avoided by the use of deep V cuts, strips of premolded asphalt filler, or open slots at the edge of the bearing area.

6. Hinges in concrete construction may be made by the use of metallic bearings. Hinges may be approximated by central or crossed reinforcement and slots that prevent bearing near the edges. However, the stress condition at such a point is uncertain. Some details as applied to rigid-frame bridges are shown in Fig. 14-21.

7. In planning any concrete bearings, special care should be taken to prevent cracking of corners near pressure points, and tensile cracks caused by undesired restraint.

13-12. Grouting of bearings. Tops of concrete piers can be poured to exact elevation and made level and smooth. However, shrinkage, cost, and practical difficulties in obtaining good workmanship with the usual labor employed make this unsatisfactory except for minor cases. The concrete may be poured high, bushhammered down to the proper elevation and condition, and the bearings set and leveled on a "paint coat" of neat cement, but this is expensive work and is usually impracticable.

More often the concrete of a pier is poured low, then a grout or mortar filler used under the bearing to bring it to the proper position. The filler, when it has hardened, must transmit the pressure to the pier. A 1:2 portland-cement mortar may be used; admixtures of various kinds may be added to reduce shrinkage; or a "rust joint" of iron filings and sulphur that tends to expand slightly may be used to form a hard pad. An allowance of 1½ or 2 in. for grout is usually a minimum. In the Bayonne Bridge, the grout was 2 ft. of reinforced concrete, as shown in Fig. 8-8.

Metallic bearings may be set and leveled by the use of wedges, as indicated in Fig. 13-52(*a*). If the mortar is soupy and poured under such a bearing of large area, it is difficult to obtain complete filling of the space.

Air pockets tend to form under the bearing, and the grout tends to slump away from the metal. The grouting is also likely to disturb the wedges. A bearing might be set on shims (or wedges), lifted off, the grout placed and made flush with the shims, then the bearing reset. This, too, may give uncertain results. Squeezing a bearing down onto mortar and adjusted shims or wedges may not work because of mortar that gets between the tops of the steel and the bearing, and the shims or wedges may be disturbed. Any such shims or wedges should be removed and the holes filled with

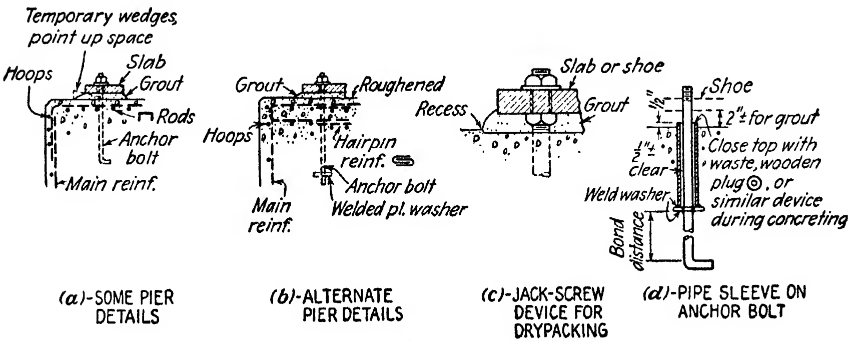


FIG. 13-52. Some details of pier reinforcement, grouting and anchor bolts.

mortar after the grout has set, otherwise the steel causes objectionable inequality of bearing. The use of centrally located holes to permit the escape of air from under the bearing during grouting is a help; emission of grout from the holes also aids one in determining that the grout has reached that area.

Dry packing of a stiff mortar by forcing it under a very heavy bearing on wedges may produce good results; it will probably disturb small bearings. The jackscrew method of threaded anchor bolts with double nuts shown in Fig. 13-52(*c*) not only enables one to level and hold a bearing, but it permits firm dry packing without chance of disturbance. However, working space must be provided around bearings to be dry-packed.

Grout may crack and work out from under a bearing. This should be prevented positively by some such method as those pictured in Figs. 13-52(*b*) and (*c*).

Grouting operations and setting of bearings require careful workmanship. In one case, some machinery bearings were to be dry-packed. Workmen of dubious skill (not under careful supervision) packed mortar around and under the edges of the bases, not getting it under the central areas. When the machines were started, the mortar failed.

Anchor bolts should be set very accurately by means of templates. The use of pipe sleeves, as shown in Fig. 13-52(*d*), permits springing of the bolts if necessary. However, if such sleeves are not filled completely with grout or melted lead after the bearing is erected, the bolts will not provide reliable shearing resistance. Furthermore, water will accumulate in the sleeves and cause rusting, and freezing may burst the pipes and crack the concrete. Pipe sleeves should seldom be used in outdoor construction.

13-13. Details of pier tops. The tops of bridge foundations are usually subjected to very large localized pressures under the bearings. The concrete can generally resist such action if the pier top has sufficient area and strength to confine the compressed region so that it cannot "go any place."

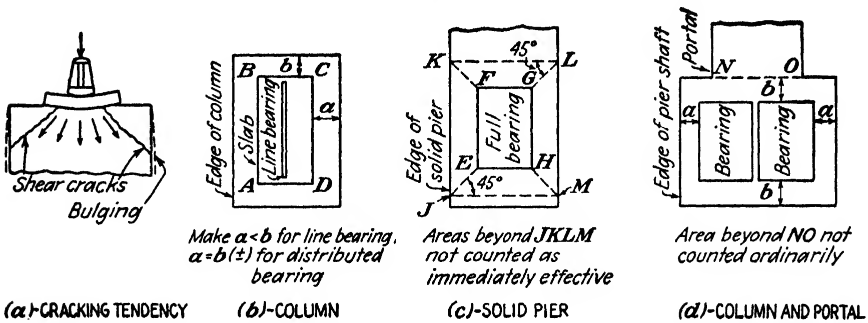


FIG. 13-53. Pier tops and positions of bearings.

That means that the shearing and bursting tendencies produced by local compression and lateral expansion (remembering Poisson's ratio) cannot crack the concrete. The general action of the localized pressure is indicated in Fig. 13-53(*a*).

A metallic bearing may be designed to produce a specified unit pressure on the concrete. However, a large steel billet under a heavy rocker will probably cause a much higher pressure under the vicinity of the line bearing than under the outer edges because of the deflection of the steel as a beam, unless plastic flow permits local yielding of the concrete. Furthermore, the localized compression in the concrete will spread out as fast as the concrete can distribute it. It seems that an average allowable unit pressure of 800 to 1,000 p.s.i. under a bearing is a desirable maximum for design purposes for any concrete of reasonably good quality. The lower figure is recommended because the prevention of cracking of the concrete is more a matter of avoiding large local deformations than of utilizing compressive strength.

The area of a pier top around a large metallic bearing should be appreciably larger than the required area of the bearing itself, or of the pair of bearings when two are adjacent. No inviolable ratio of the areas can be stated. The top of the pier should be planned to provide a reasonable area, to look well, and to give maintenance men a chance for a foothold when working around the bearings. The shape of this area with respect to that of the bearing is also to be determined with good judgment. However, if other controlling factors are absent, the arrangements illustrated in Fig. 13-53 are suggested. The following descriptions refer to the corresponding sketches:

(*b*) This pictures a single rocker bearing on a rectangular shaft. Most such bearings are longer in a direction transverse to the bridge than in that parallel to it. If the top of the pier is very large for architectural or other reasons, try to have the excess area symmetrically placed with respect to the bearing.

(*c*) This shows a massive rectangular pier with a single cast or welded shoe. The minimum edge distances a and b are again determined by judgment, and portions outside of $JKLM$ are classed as temporary excess. Of course, this does not mean that the latter areas resist no load as the compression spreads downward in the pier; they are assumed to have little beneficial effect at the top.

(*d*) This assumes a pair of bearings on top of a two-column portal-braced shaft. The edge distances a and b may be determined as for a single bearing. The area of the top of the portal is not assumed to be effective, although it might be so if designed for the support of an intermediate bearing.

The preceding suggestions are given in order to prevent the placing of bearings on unduly slender shafts or close to the edges of long narrow pier tops where two edges will be compressed heavily whereas the areas on the other two sides are not very effective. This may seem to require piers of large cross-sectional area. It does, and they are usually so for other reasons: to provide an appearance of strength and proper proportion. If more slender piers are desired, they should be designed somewhat as columns, with special provisions to prevent harmful cracking of edges.

Reinforcement of the top of a pier to prevent cracking from localized loads is helpful in the avoidance of visible cracks and spalling. It is not very effective in spreading loads. It cannot offer resistance to tensile forces except as there is deformation—and this may be when hair cracks have formed. The steel is also rather ineffective as dowels to transmit shearing stresses. The schemes of reinforcement shown in Figs. 13-52(*a*) and (*b*) are suggested. However, they are designed primarily to hold the top corners and edges in place even if tiny cracks do occur.

If the depth of a coping is four to six times its projection from the body of the pier and if it is reinforced properly, the area of its top may be assumed to be the top of the pier when considering the area of pier vs. bearings. However, no overloading of the shaft should occur under the coping.

It is generally desirable to slope the top of a pier away from the bearings sufficiently to ensure good drainage.

13-14. Miscellaneous items. A facing of granite masonry on that portion of a pier in and a few feet each side of the tidal range constitutes a good protection against deterioration caused by sea water and freezing. However, such masonry must be held adequately. A thin veneer applied after the concrete has set is usually a waste of money. Heavy stones bonded and anchored to the concrete are essential. Some details of such construction are shown in Fig. 13-16.

If wooden guards or other objects are to be fastened to bridge substructures, noncorrodible bolts should be embedded in the concrete. It is even better if through pipe sleeves are used so that long bolts can be inserted and later removed when desirable or necessary. Plain steel or galvanized bolts rust off surprisingly fast, especially in and near salt water.

The drains from roadways should be located so that their discharge will not cause staining of the substructure. Drainage pipes extending down through the concrete to or near the water line may become clogged with debris or closed by icicle action during cold weather. If such pipes are used, they should be of large-sized cast iron (or formed holes), and they should terminate in a projecting 45° bend well above the water line. There should be special provisions for protection and drainage at joints in the roadway, somewhat as shown in Fig. 14-35.

Adequate provisions for drainage and for the avoidance of unsightly stains are also essential in such cases as the intermediate piers in a series of spandrel-filled barrel-type concrete arches. Rain water will almost inevitably get through the pavement and tend to collect at the piers as though in a sump. After a time it will seep through construction joints or the concrete itself, causing efflorescence, stains, and even spalling.

What reinforcement should be used in a concrete pier? As an example, assume the simple construction shown in Fig. 13-41. This is just a thick concrete wall that transmits loads in compression from the superstructure to the ground. It is not an anchor pier that is subjected to large longitudinal bending as a vertical cantilever; neither is it a vertical beam that participates in bending caused by rigid-frame action of the structure. Since the unit compression is also small, vertical rods are structurally unnecessary. Shrinkage and temperature changes will not cause the pier to "jump up" into the air; nor will loads on the structure do so.

If the pier is long, horizontal steel may be useful in preventing visible vertical cracks resulting from chemical or thermal shrinkage because the friction on the bottom may not let the wall shorten easily. This may lead to the use of a few bars, perhaps an area of steel equal to 0.0025 times the vertical cross section of the pier. Then a few vertical rods will naturally be used to hold up the horizontal ones prior to pouring. These should be at least ¾-in. rounds in order to have sufficient stiffness. They might be used 3 ft. on centers. A layer of bars near each face of the pier is better than a single set at the middle. The rods may then be of some help in preventing major spalling of the concrete, although spalling that goes deeper than the usual 2 or 3 in. of cover over the steel means that the pier looks like a wreck anyway. Good quality of workmanship and the use of dense concrete are the best means of preventing bad spalling.

In a pier like that of Fig. 13-22(g), the frame action of the portal and strut, or rib, will ordinarily require some reinforcement in them and in the neighboring portions of the shafts, at least near the junctions. Cantilever action at the junction of pier shafts and footings may also make the use of reinforcement desirable here. It is then sensible to continue some reinforcement for the full height of the pier. In many cases, however, this reinforcement seems to be overdone. In brief, it should be used where it serves a useful purpose, but it should not be wasted.

When reinforcement of the junctions of heavy concrete construction, like the portal of Fig. 13-40, with the pier shafts is planned, the steel should be located at edges, corners, or other places where cracks may start. Other rods scattered in between will not be very useful.

When a long pier is built on rock, it is not free to shrink because of bond or mechanical gripping of the rock. Horizontal steel then merely helps to compel cracks to be tiny unnoticed hair cracks instead of localized large ones.

So-called surface reinforcement below tide and splash level of sea water may actually accentuate spalling if the concrete is sufficiently porous to permit the rods to rust badly and thus "pry" off the concrete over them.

In the approach piers of the Bayonne Bridge, which are somewhat like those shown in Fig. 13-40, structural-steel towers were used as reinforcement. These greatly facilitated construction because they could be erected as units, equipment could be supported upon them, and the placing of rods was eliminated.

It may seem economical to make pier shafts hollow in order to save concrete and to minimize weight. The former may not result in economy because of the cost of the inner forms. If the hollow portion is extended below the water line, seepage may cause water to collect in the void, then freezing of this water may crack the pier. The hollowed portion should stop above water and should be drained.

If a pier or steel bent for a viaduct is alongside a railroad track, it is desirable to plan the construction so as to resist derailment. No magnitude is specified for the force to be resisted, but the shape and strength of the construction should be such that the safety of the structure is assured. A colliding car should glance off the side of the structure, not catch upon projections, hence a smooth streamlined contour is preferable. If the pier consists of steel columns, these should be based upon a concrete wall 8 to 12 ft. high, or should be encased in a concrete wall up to this height. A wreck is bad enough without having collapse of the structure add to the disaster.

Small projections on concrete construction are undesirable because of their tendency to crack off. Projecting corners should be beveled. Grooves or V cuts may well be used at construction joints to conceal them.

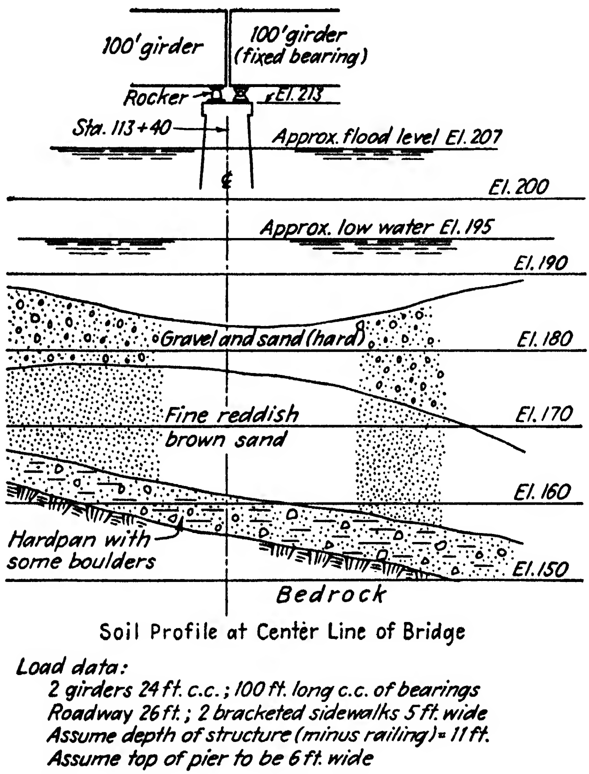


FIG. 13-54. Data at a pier for a deck girder bridge in Connecticut.

When such markings are a part of the architectural treatment, they should be sufficiently broad and deep to cast shadows that will produce the desired effect. The same comment applies to recessed panels.

Considerable discussion has occurred regarding the question of whether the sides of a pier should be battered. Of course, batter increases the difficulties with forms, but it is intended to give the impression of strength—increased area near the base for a structure cantilevered from the ground. It seems appropriate for massive solid piers. It is less so for those that are primarily independent or braced columns. The use of steps or projections as in Figs. 13-22(*c*) and (*g*) is another method of attaining the general effect of a reduction of area with height.

Copings are questionable features. With stone masonry they were appropriate as a top finish and a means of protecting the top joints from weathering. They are still advisable for such masonry and for stone-faced concrete, whether the copings are of stone or concrete. Stone is preferable. For concrete structures there seems to be no real or imaginary need for copings. It is better to design concrete structures as honestly made of concrete, not as an imitation of something else.

Special care is needed to obtain properly formed surfaces free of waves, offsets, fins, honeycombing, and streaks. Unusual surface treatments may be desired in certain cases, but a clean plain surface is usually preferred. Straight troweled edges at construction joints, or V cuts as pictured in Fig. 13-33, often add much to the appearance of a pier.

A discussion of piers for movable bridges has been omitted purposely. These are generally special piers planned to suit the particular superstructure that each is to support. They are generally low and broad. Their design involves the same general principles as those described for other piers.

In general, the shape and details of a pier should be such that the completed structure gives the observer the idea that it is simple and functional, neat and attractive, strong, and suitable for its purpose.

PROBLEMS

13-1. Figure 13-54 pictures the soil conditions found at the site where a pier is desired. It is in glacial territory. Ordinarily the stream has a moderate current, as

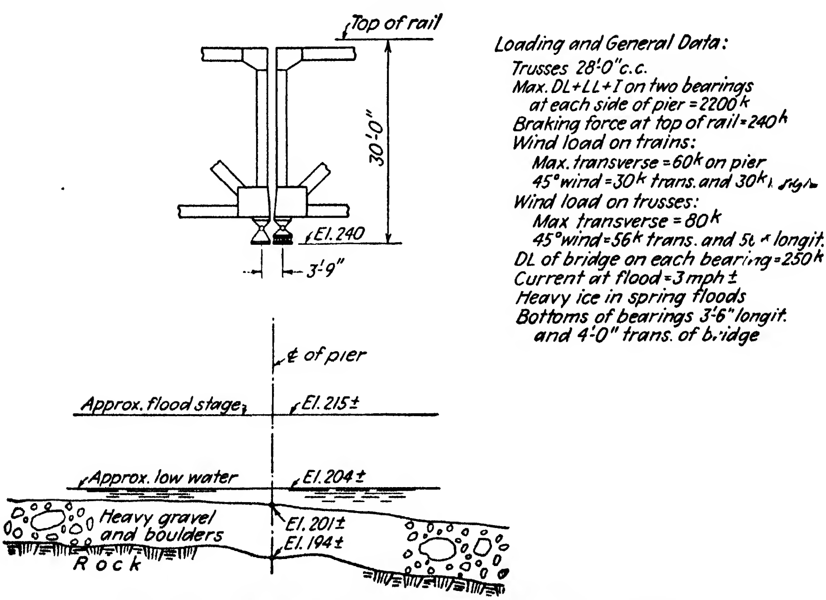


FIG. 13-55. Site for a railroad bridge pier in a shallow, rapid river.

shown by the gravelly bottom. Design a pier that is suitable for this structure. Assume the loads from the data given in the illustration and by use of Figs. 13-34 to 13-38.

Suggestions: There is some drifting ice in the spring breakup but this is not severe in Connecticut. Consider scour. Is the gravel sufficient to confine the sand? If the pier is founded upon the sand, will it be safe?

13-2. Design a pier for the railroad bridge and conditions pictured in Fig. 13-55.

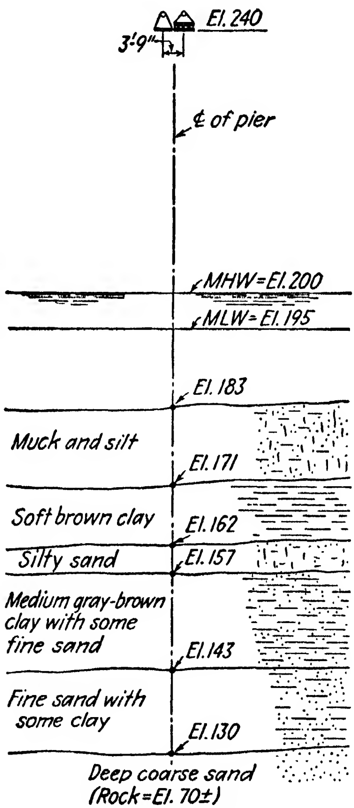


FIG. 13-56. Site for a railroad bridge pier in a tidal estuary.

13-3. Figure 13-56 shows the soil profile at the site of a pier to be built along the Atlantic Coast. Assume the same superstructure and loads as shown in Fig. 13-55, except that there is little or no ice and current.

Suggestions: The longitudinal loads may now become very serious. Assume allowable bearing values of soil or loads on piles that seem to be consistent with the soil data. Do this before designing the pier, then adhere to the original assumptions and see what are the results.

14

BRIDGE ABUTMENTS

14-1. Introduction. A bridge abutment may be defined in general as the foundation that supports one terminus of the superstructure of a bridge. Usage varies somewhat but this is not important. The distinction in character between a pier and an abutment will be obvious as the reader proceeds with this chapter. Naturally, however, a bridge must have a beginning and an end when considered by traffic using it, and these will be called the abutments.

Modern bridge abutments are generally made of reinforced concrete. Stone masonry may be used occasionally, but it is ordinarily incorporated as a facing backed with concrete, this being the case when special architectural effects and durability of surface are desired.

Much of the information given in the preceding chapter is also applicable to the planning and design of abutments; *e.g.*, scour, bearings, forces applied by the superstructure, and many details. These things will not be repeated but are taken for granted.

If the type of abutment to be used at a given site is not automatically determined by architectural considerations, the owner or the engineer may have a personal preference based upon past experience, or upon what has been used in a somewhat similar case. However, in any situation, safety, economy, and the practicability of construction are always important. Alternate designs and estimates are generally justified in the case of large structures.

The dimensions and character of the superstructure have considerable influence upon the type of abutment that is most suitable in a given case; so do local topography and subsoil conditions. In this chapter, therefore, an attempt has been made to discuss the planning and design of abutments somewhat in accordance with the type of superstructure in terms of the materials used or the structural action of the bridge. The illustrations may be helpful in aiding one to develop a plan that will be satisfactory in all respects.

As with other foundations, the engineer should try to visualize how his structure might fail, then try to make sure that it will not do so. An abutment may settle unevenly because of the larger pressure caused at the toe by the earth behind the abutment and by the heavy bridge load in front. If the earth under it is not uniform, the abutment may tilt sidewise. It may even slide forward into the water as a whole, or the bottom may skid forward while the top tilts backward. Such are the important things to guard against, and they may be troublesome. An abutment as a structure is usually so large that there is little difficulty in making it sufficiently strong and stiff of itself. Keeping it where it belongs may be more difficult.

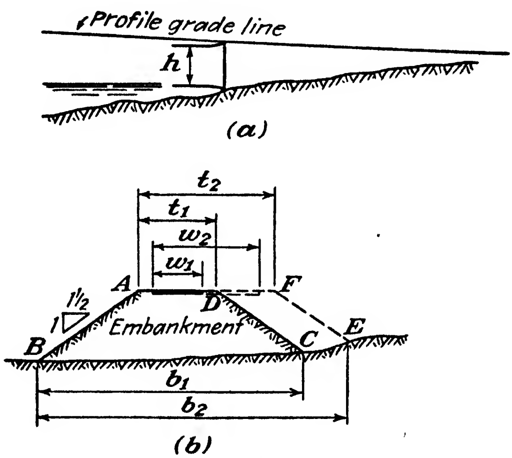


FIG. 14-1. One of the features to consider in the location of an abutment is the relative cost of fill and superstructure.

14-2. Planning locations. The determination of the locations of the abutments of a bridge is only one of the problems incident to the general planning of the structure. In some cases, an abutment may be a minor part that merely holds the end of one span of the approach portion of the bridge; in others, a large and costly foundation that supports one end of a major unit of the superstructure.

It may be that an abutment is located at a point where local conditions make it almost automatic. Again there may be considerable latitude for the selection of its position. The following are a few of the considerations that may be important in establishing a position:

1. An abutment may be placed at the point where an approach embankment becomes so high that the use of elevated structure is more economical than a continuation of the embankment. This is not very definite. In general, if the height h of the fill, as shown in Fig. 14-1(a), is 20 ft. and if a continuation of the embankment would cause h to increase considerably

and progressively, then it is probably desirable to consider seriously the use of superstructure rather than a continuation of the embankment. This, however, may depend upon the width of the roadway and the magnitude of the live loads.

As indicated in Fig. 14-1(*b*), a roadway of width w_1 may require the fill *ABCD*. If the width is doubled to w_2, the fill *ABEF* is by no means double that of *ABCD*, whereas the cost of the superstructure will be much more nearly increased as w_2/w_1. This would indicate that higher fills may be relatively more economical for wide roadways than for narrow ones.

On the other hand, a fill is seldom more costly when it supports heavy loads than when it has light ones (it should be well compacted anyhow). But this is not so for the superstructure. Thus, it may be more economical to use higher embankments for railroad construction than for highways.

2. Soft clay or silt may be so compressible, and subsequent settlement or lateral flow under the weight of a large fill may be so serious, that it is advisable to extend the superstructure much farther than would be done otherwise in order to avoid the use of a massive embankment.

3. If the layout of roadways is such that a wide flaring area is required for side connections, even high fills may be preferable to costly complicated elevated construction. Thus the abutment may almost automatically be placed at the point where the roadways merge into the width of the main bridge roadway, or at least where the flare is small.

4. A transverse highway or railroad that is to be overpassed by the new structure may be a natural obstacle that limits the abutment to a position

FIG. 14-2. The Henry Hudson Bridge across the Harlem River at New York City. This is a double-deck highway bridge with fixed-end arch ribs. *(Courtesy of D. B. Steinman, Consulting Engineer, New York, N.Y.)*

just beyond the landward (outer) right-of-way line if there is vertical clearance enough to permit the superstructure to pass overhead. If the clearance is inadequate, or if the horizontal position proves to be an obstacle, the main structure may have to terminate beyond the waterward side, making it necessary to use a special flanking structure over the road or tracks. Such a situation establishes one control point in the planning. This is illustrated by the effect of Riverside Drive on the

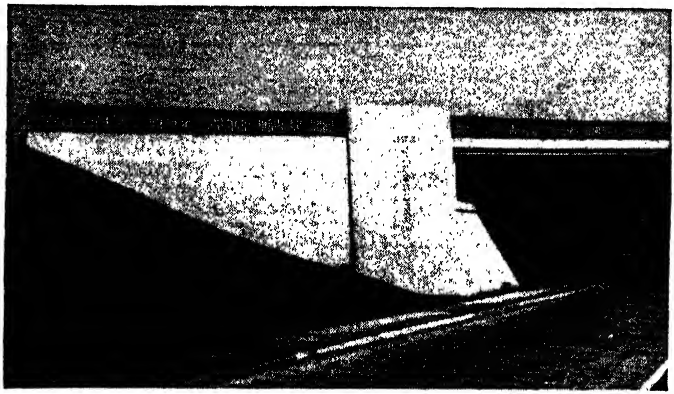


FIG. 14-3. A multigirder steel bridge carrying the Wilbur Cross Parkway over Whitney Ave., Hamden, Conn. The bridge is on a skew, and the wings of the abutment are to accommodate this. *(Courtesy of the State Highway Department of Connecticut.)*

location of the New York anchorage of the George Washington Bridge, Fig. 14-34.

5. Symmetry, or at least reasonable similarity of both sides or approaches of the superstructure, is generally desired for the sake of appearance. Therefore, when the abutment and piers of one approach to a main span are located to meet special requirements, it may be desirable to use similar positions for those of the other approach unless special local conditions there prevent.

6. Local good soil conditions may make it important to locate a heavy abutment so as to take advantage of them, whereas adjacent weak soils are to be avoided as much as possible.

7. Sometimes, the topography and the presence of sound bedrock leave almost no question as to where the abutments ought to be. Figure 14-2 illustrates this.

8. Horizontal and vertical clearances that must be maintained are likely to cause the abutments to be as near the limits of the former as the shape and type of structure permit. This is often demonstrated by rigid-frame and other highway overpasses, such as those of Fig. 14-24. In the case of the Bayonne Bridge, Fig. 14-28, the upper portion of the government's

clearance diagram for the channel limited the position of the roadway structure. Then the general shape of the arch fitted over this, and the abutments had to be placed where needed to hold the structure.

9. The possibility of damage by scour, shore erosion, ice, and floods may cause the abutments to be located back out of danger, even though this causes the superstructure to be longer.

10. When an abutment is to be of some particular type and to present a certain architectural appearance, then its location may be determined

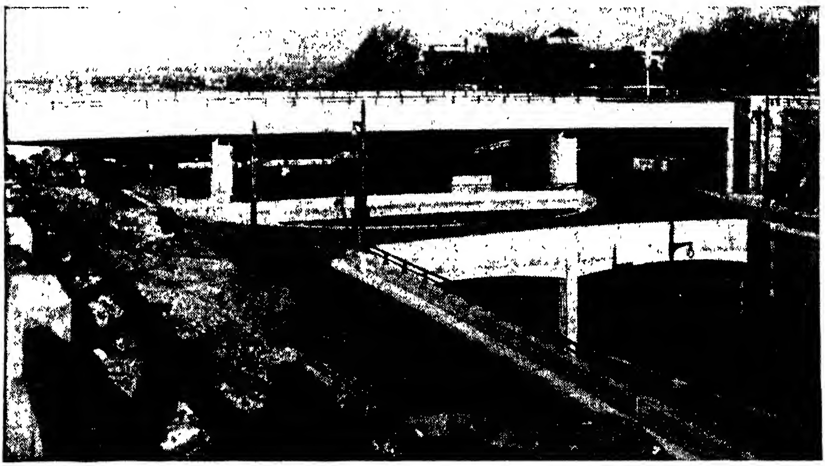


FIG. 14-4. Three-level traffic interchange at Hudson County Boulevard West, the New Jersey approach to the Lincoln Tunnel. The lower structure is a two-span reinforced-concrete rigid frame. The upper one is structural steel encased in concrete. *(Courtesy of the Port of New York Authority.)*

largely by the position where it will have the proper size, height, and setting in the topography to produce the desired aesthetic effect.

There are many cases in which the best locations for the abutments of a bridge are very uncertain. This is particularly true in the case of stream crossings when such conditions as the following exist:

1. Wide streams with large areas of shallow water near shore.
2. Streams with extensive marshy areas along the shores.
3. Streams with low, flat, or gently sloping ground for long distances back of the shore line.
4. Streams which frequently flood wide areas of adjacent low shores and for which the area of waterway should not be constricted.
5. Streams which meander in soft alluvial soils and which may change radically the locations of their channels.
6. Railroad bridges and low-level bridges whose approaches have little or no grade and which remain nearly parallel to the ground for considerable distances.

For such cases, an engineer will probably locate the abutments where the resultant total cost of the project is a minimum, or wherever, in his opinion, they ought to be.

14-3. Abutments for simply supported and continuous bridges. Seats for the bearings of steel and concrete bridges are to be provided as in the case of piers. This is one of the starting points in the planning of an abutment. The area of the seat should be adequate, and the bearings of steel bridges should be so placed that they can be inspected and maintained

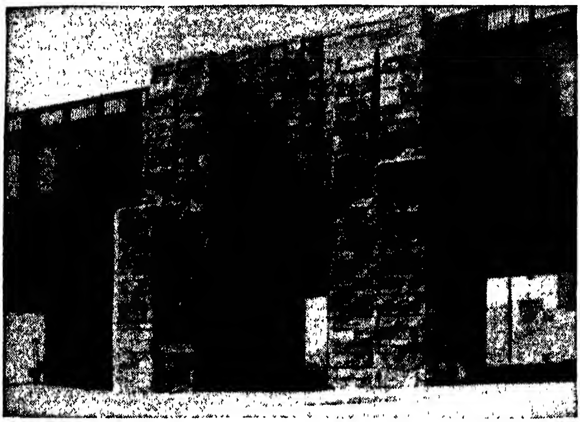


FIG. 14-5. Abutment between two steel girders of the New Jersey approach to the Lincoln Tunnel. This shows an attractive treatment with stone facing and shielded or recessed bearings. *(Courtesy of the Port of New York Authority.)*

easily. There may be special cases like that of Fig. 14-4 where the architect preferred that the bearings be hidden. Here the steel girders supporting the boulevard are encased to make them appear as though they were concrete construction, and the bearings are in pockets which are covered by removable steel plates attached to the girders and which slide over the tops of the concrete walls around the pockets. Another instance of concealed bearings is shown in Fig. 14-5.

Consider first some of the ordinary types of abutments used with steel and concrete bridges other than arches and rigid frames. The details of construction vary widely in practical cases to suit the local conditions. However, for convenience, various abutments will be grouped and named in accordance with some basic principle underlying the design of each. The drawings are simplified to show essential features only.

1. Straight-wing abutments. Figure 14-6 shows two abutments of this type. In general, they are retaining walls modified so as to support the superstructure. They are used with embankments of moderate height. They are often built at stream crossings where the wings prevent the fill from blocking the stream, and where they tend to restrict the scouring action of water eddying around the main support. In the case of underpasses, the wing walls may actually be replaced by long retaining walls when necessary. This type is adaptable to use with skewed as well as with

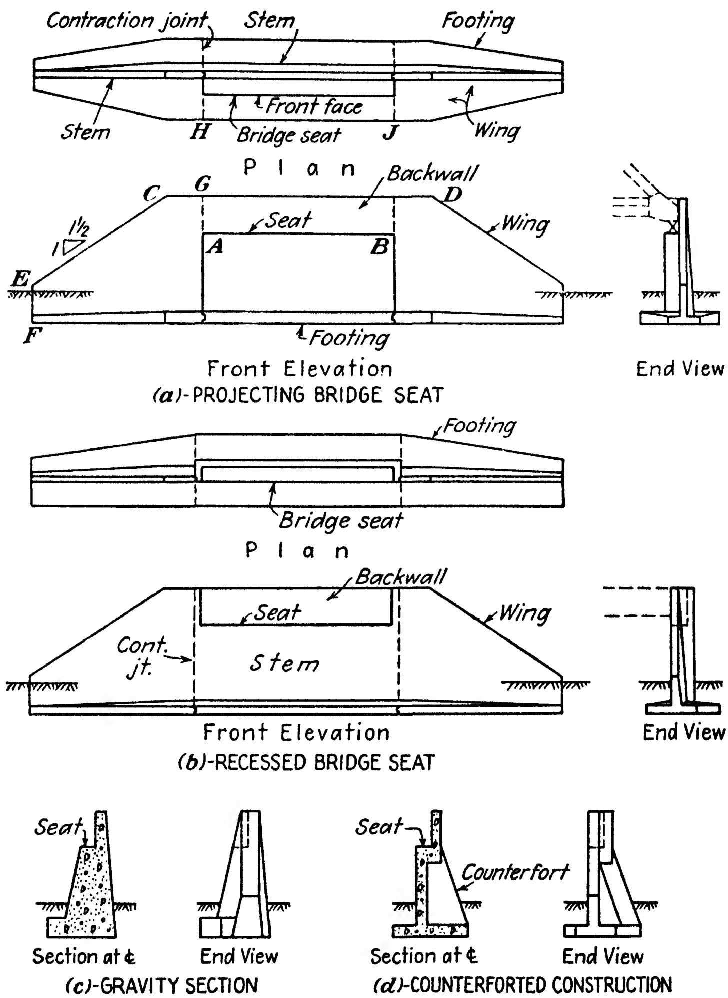


FIG. 14-6. Straight-wing abutments.

normal bridges. Such abutments are usually massive and must resist large overturning moments. Therefore, they are suited best to use on sand and firm soils.

Sketch (*a*) shows a continuous projecting bridge seat. This is simple construction. The bearings are exposed, and the backwall is a continuation of the wings. With through trusses, pony trusses, through girders, and deck trusses, pilasters may be used under the bearings only instead of having a continuous seat. This, however, because of overturning, tends to cause large local pressures under the toe of the footings if they project locally in front of the pilasters. Even with a continuous bridge seat, beveled or offset footings of the wing walls tend to concentrate the pressure at *HJ*.

In Sketch (*b*), the bridge seat is recessed by making the wing walls flush with the front face of the central structure. This may be a preferred arrangement for underpasses where the cut for the lower roadway is to be

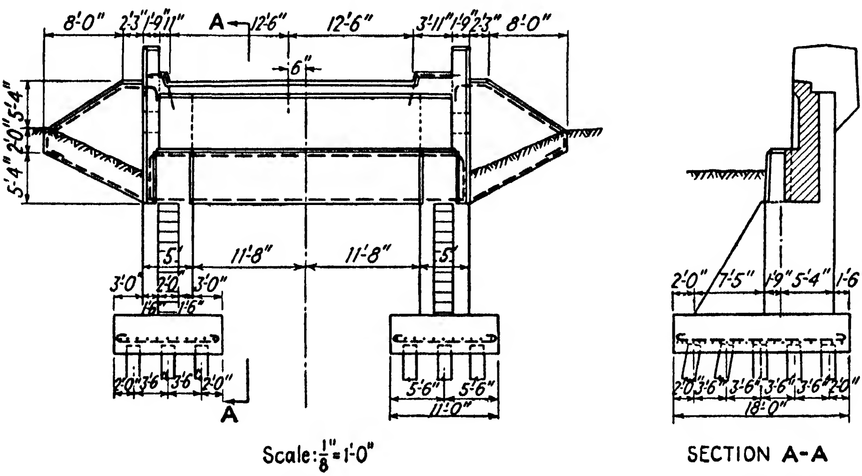


FIG. 14-7. An abutment on two pile-supported piers. The shallow backwall and cantilevered wings minimize the horizontal earth pressure that the structure must resist. *(Courtesy of Parsons, Brinckerhoff, Hall, & Macdonald, New York, N.Y.)*

minimized, for cases where the wings are needed to distribute heavy bridge loads over large footing areas, and where the toe is to be in one line in order to minimize the critical bearing pressure at the front edge.

Such abutments are inclined to be long. The bridge seat *AB*, in Fig. 14-6(*a*), is dependent upon the width of the superstructure. This may be 40 to 60 ft. or more. The corners *C* and *D* should be somewhat beyond the limits of the shoulders of the fill—perhaps 6, 8, or 10 ft. beyond the edge of the roadway pavement. The slope *CE* of a wing wall depends upon that of the embankment. The distance *EF* will be determined by frost depth, and by embedment to reach firm soil or to guard against scour.

Figure 14-7 shows a very different design but one that, above ground, appears to be similar to the heavy one of Fig. 14-6. The backwall and bridge seat constitute a heavy beam spanning between buttressed piers on piles, and the wings are its cantilevered extensions. This is a far more economical design when supported upon weak soils. It may be used advantageously on good soils if considerable depth is needed to reach firm ground or to avoid danger of scour. The basic idea may well be borne in mind in connection with other types of abutments.

If contraction joints are not provided in such a long concrete structure, nature will usually create them because of shrinkage and temperature action. Therefore, joints may be advisable, as shown in Figs. 14-6(*a*) and (*b*), where the section changes abruptly. If the seat is over 40 ft. long, a vertical joint may also be advisable at its center. In general, all such joints should be keyed to hold the parts in line; those in the stem should be

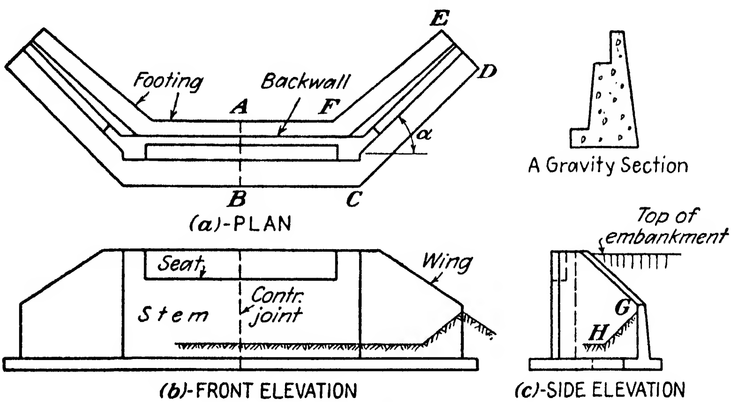


FIG. 14-8. Beveled-wing abutment.

flashed to prevent leakage and staining. When the length is small enough and when the wings are to be used to spread bridge loads, it may be desirable to have a joint at the center of the abutment only. In other cases, when the abutment is high, keys may be used in the footings but not in the stem, thus permitting the wings to deform differently as thin retaining walls than would the massive central portion. Steel bridge construction will generally permit the tiny movement at contraction joints without harm to the superstructure. In the case of concrete slabs and T-beam construction, one should take precautions to avoid having a contraction joint in an abutment cause a corresponding crack in the bridge deck.

In general, skimpy highly reinforced sections are inadvisable for such abutments. Parts should be strong, rather massive, and stiff. However, adequate horizontal reinforcement and vertical contraction joints are essential to prevent cracking.

2. Beveled-wing abutments. In some ways, the construction shown in Fig. 14-8 is a minor modification of that of Fig. 14-6. However, it has considerable difference in structural action when the superstructure is narrow and the angle α of the wings is large, perhaps 30 to 45°. This is because the entire abutment is of a shape that is basically stable; *i.e.*, the wings tend to serve as counterforts so that the structure is stronger than would be a straight T-shaped retaining wall. When the length requires a contraction joint at the center, a portion like $ABCDEFA$ still has a considerable resistance to being tipped over as a unit. Of course, if contrac-

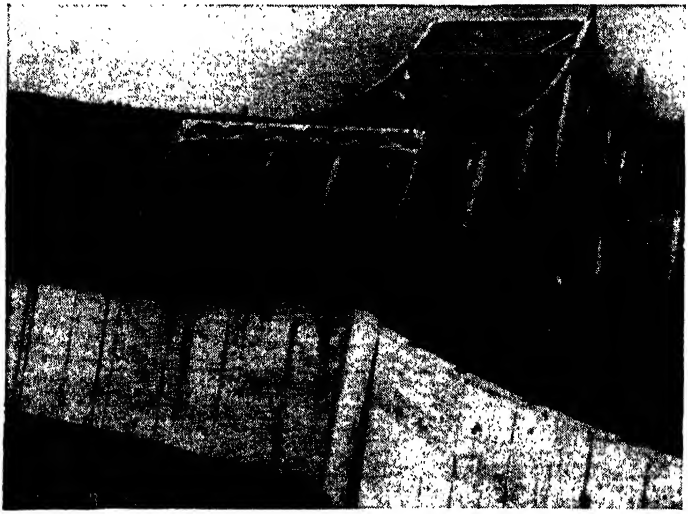


FIG. 14-9. Abutment at the Glassport end of the Dravosburg Bridge over the Monongahela River in Allegheny County, Pa. Notice how the wing wall is curved to accommodate a connecting highway. *(Courtesy of Parsons, Brinckerhoff, Hall, and Macdonald, Consulting Engineers, New York, N.Y.)*

FIG. 14-9A. A plate girder bridge carrying the Montour Railroad over the relocated McClarens' Run in Allegheny County, Pa. Notice the skew, the beveled wings, the walls protecting the bridge seats, and the handplaced stones to protect one side of the cut. *(Courtesy of Parsons, Brinckerhoff, Hall, and Macdonald, Consulting Engineers, New York, N.Y.)*

tion joints are used in the vicinity of CF, each part must then stand up by itself as a retaining wall.

A beveled-wing abutment may be used at a crossing over a stream when the wings are useful to deflect the stream toward the opening, provided serious scour is not probable. Sometimes such a structure may seem to fit the terrain better and to be more attractive than one with straight wings.

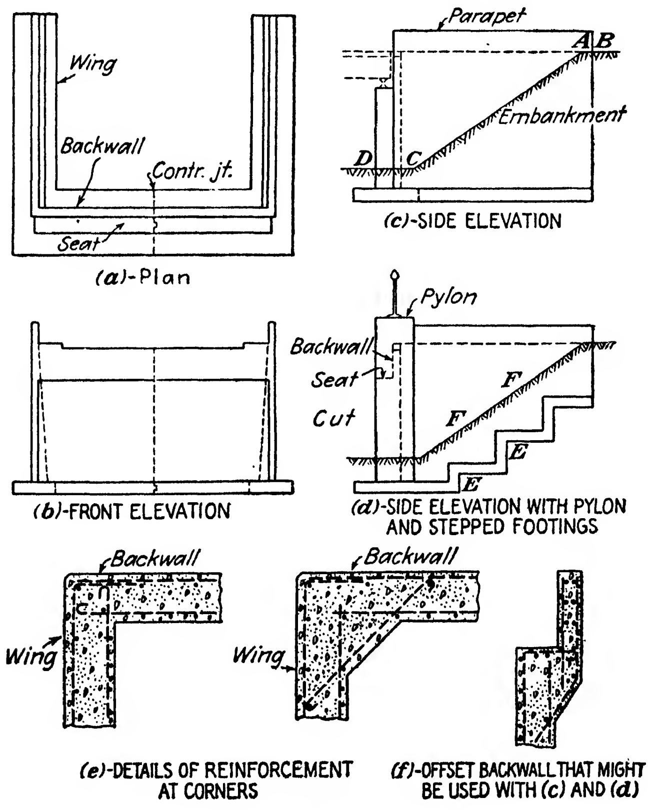
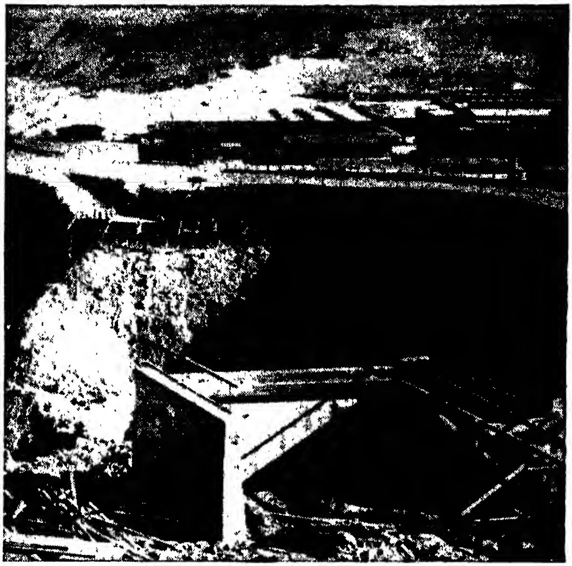


FIG. 14-10. U-shaped abutment.

This type may be easily adapted for use with skewed structures. It may reduce the fill somewhat, but this is seldom important. When the fill may be allowed to run around the end of the wing as shown by GH in Sketch (c), the amount of concrete required may be reduced a little. In general, such an abutment is heavy and is most suitable on firm soils.

3. U-shaped abutments. The construction shown in Fig. 14–10 has the wings perpendicular to the front face. They serve as effective counterforts if the bridge is not too wide. Therefore, this is an inherently stable structure when it is designed and founded properly. The illustration indicates



(a) These abutments are supported upon a very deep fill. They are designed with a wide base to reduce unit loads but they are expected to settle slightly with the embankment. No attempt is made to hold back the main fill itself.

(b) A concrete roadway slab is to be placed on this steelwork. There is to be a joint in the roadway above the far side of the tower in the background so that the end span can rotate about a pin on top of the post if the abutment and fill settle.

FIG. 14-11. A steel highway bridge with steel towers as piers, and with small U-shaped abutments at the Morenci Reduction Works, Morenci, Ariz. *(Courtesy of the Phelps Dodge Corp.)*

what might be used for supporting a deck highway structure composed of heavy beams or girders placed longitudinally under the roadway.

Such an abutment is advantageous for grade-crossing eliminations in rural areas where the sides of the cut for a lower roadway are to be sloped to avoid the use of the costly retaining walls that are often necessary along such cuts in urban areas where land is valuable. In other cases, they constitute a desirable terminus for the embankment of the upper roadway because they appear strong, as well as being so. This visual effect is often advantageous as a suitable end and support for the superstructure. This type is not so advantageous for crossings that are skewed sharply.

When the upper roadway is on an embankment, the wings may be built as in Fig. 14-10(*c*) so that the footings are on good undisturbed soil, and the fill is allowed to slope along the outside. It is generally desirable to have the overlap distance *AB* at the top of the embankment at least 2 ft. in order to prevent settlement and erosion from making the end of the wall seem to be cut too short, and in order to support the fill just beyond the end. The distance *CD* beyond the toe of the slope should also be sufficient to avoid the appearance of weakness and skimpiness.

When the lower roadway is in cut, the wings may have stepped footings somewhat as pictured in Sketch (*d*) because it is not necessary to excavate deep trenches back into the undisturbed ground and in them to build walls that have to resist little difference in lateral pressure. Of course, the bearing resistance of the soil at such corners as *E* may be largely imaginary, but there will generally be enough resistance under the rearward portion of each step to hold the structure. The embedment *EF* should exceed the expected frost depth even though natural drainage may prevent the formation of harmful ice lenses. The rearward portion of the footing may be made parallel to the slope instead of stepped, if the slope is flat enough for the soil utilized. This, however, may require undesirable top forms when the footings are poured.

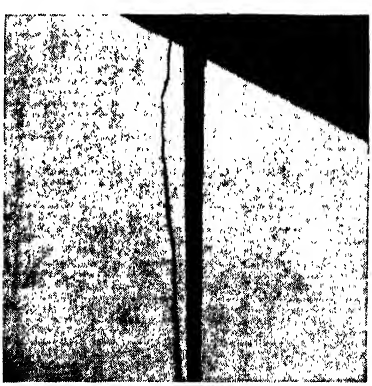


FIG. 14-12. A crack in a large bridge abutment. This seems to be the result of shrinkage.

A central contraction joint is desirable in such an abutment because the embankment fills the inside of the U. Such fill is assumed to be used with this type. After the fill is placed, traffic tends to compact it against the three sides. When the temperature drops, the concrete cannot shrink without tending to squeeze the warmer fill inside, and the latter's abutting power will usually exceed the tensile or shearing strength of the abutment, as illustrated in Fig. 14-12.

If the abutment is so wide that one central joint is not sufficient, one must be careful not to locate the joints where they destroy the mutual counterfort action that is so beneficial. For example, clear joints through the wings at their junctions with the front and parallel to it compel each of the portions to act by itself as a retaining wall; shear keys in these joints will almost surely be broken by the shrinkage action, and they do not permit the wings to hold the front against tipping outward. If two joints are needed, as in an abutment 60 ft. wide, they might be keyed joints through the front or bridge-seat portion and located about 10 ft. from the junction with the wings. The keys can then be used to enable the angular end portions to help steady the middle piece. The wings and the front should be strongly tied together at their junction.

In some cases, concrete may be saved by the use of an offset backwall, as shown in Fig. 14-10(*f*). In others, pilasters may be used at the bearings only, constituting buttresses for the front wall. Again, extra concrete may be used deliberately in order to attain a desired architectural effect, as illustrated by the pylon indicated in (*d*).

In any event, U-shaped abutments should be and can be made attractive in appearance. V-cut markings, paneling, special form finishes, pilasters and pylons, attractive balustrades or railings to reduce apparent height, horizontal offsets, and fluting—these may yield excellent results without much added expense when they are used expertly. Fancy gingerbread used on such concrete construction as camouflage is generally unwise; stone-masonry facing on monumental structures may produce such attractive results that it is worth its cost.

4. Box abutments. Outwardly, the abutment shown in Fig. 14-13 may appear to be U-shaped because of the wings. It is, however, a partial box resting upon the ground. The one shown here might be used under a through plate girder or truss bridge.

The two pilasters are practically piers on spread footings. The backwall behind the bearing is extended down as a curtain and may be utilized to help spread the bridge loads. Part of this wall might be omitted, but its use is desirable in order to avoid recesses that might become filled with debris. Sometimes, a temporary opening may be left in it for the removal of forms. The wings are also curtain walls that may or may not have footings. At the rear, a secondary wall and footing are supported upon undisturbed ground near the top of the slope, or they may be placed on piers as shown in Sketch (*c*) by dotted lines when the upper roadway is on embankment. The roadway deck is an integral part of the abutment. It and the wings span from the front to the secondary wall. When the bridge is narrow, the deck and secondary wall might span crosswise between well-supported wing walls.

A box abutment eliminates the fill that required heavy retaining walls in the preceding cases. It may, therefore, reduce the weight, and thus be suitable for use on weaker soils while appearing to be massive. Of course, the structural deck costs money. With wide structures and deep cuts, this may offset other savings.

Referring to Fig. 14-13(c), the rear footing should be placed far enough back from the edge of the slope so that the soil can spread the loads prop-

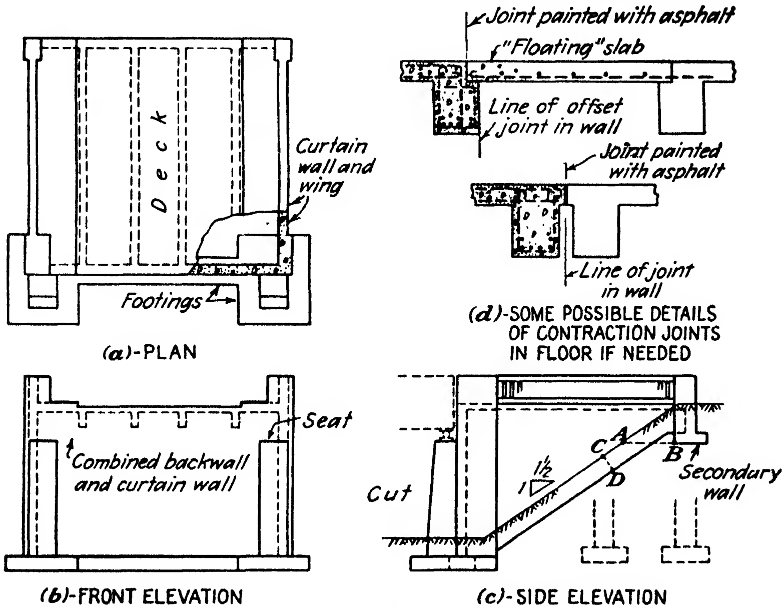


FIG. 14-13. Box abutments.

erly. In general, AB should be at least 5 or 6 ft. If the wing walls span from front to rear, the embedment CD may be 1 or 2 ft.—enough to make sure that erosion does not expose the bottom.

It is usually best to have a box abutment built as an integral structure. If the front is over 40 ft. wide, it may then be desirable to have a keyed contraction joint along the center line. Furthermore, a transverse row of intermediate piers may be used to support the center of the deck when the latter is too long to be economical as a single span.

It is obvious that a box abutment may be skewed readily when the angle is not too sharp for proper appearance. It is a type worthy of study when the cut is deep or the fill is high.

5. Flanking-span abutments. It is logical in many cases to carry the idea of reducing the lateral pressures on abutments and of securing economies still further. Figure 14-14 shows this in what will be called a flanking-span abutment. Here, the wing walls are frankly omitted; when feasible, the front wall may also be reduced by the use of rectangular or arched openings.

This abutment reduces the weight to that of the structural essentials, yet it is automatically sturdy. The deck and secondary wall steady the structure longitudinally; the front should be sufficient in frame action or in

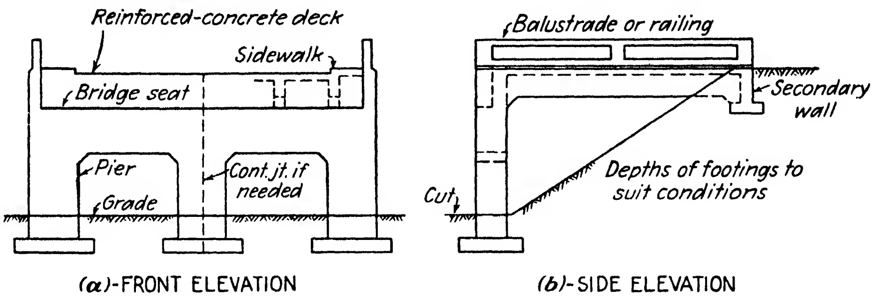


FIG. 14-14. Flanking-span abutment.

resistance to overturning to provide transverse strength without depending upon the deck to resist twisting. Naturally, contraction joints must not be used where they will interfere with the structural action. If the bridge is very wide, the abutment might then be built somewhat as two abutments side by side. Although this type may be better than many others for use

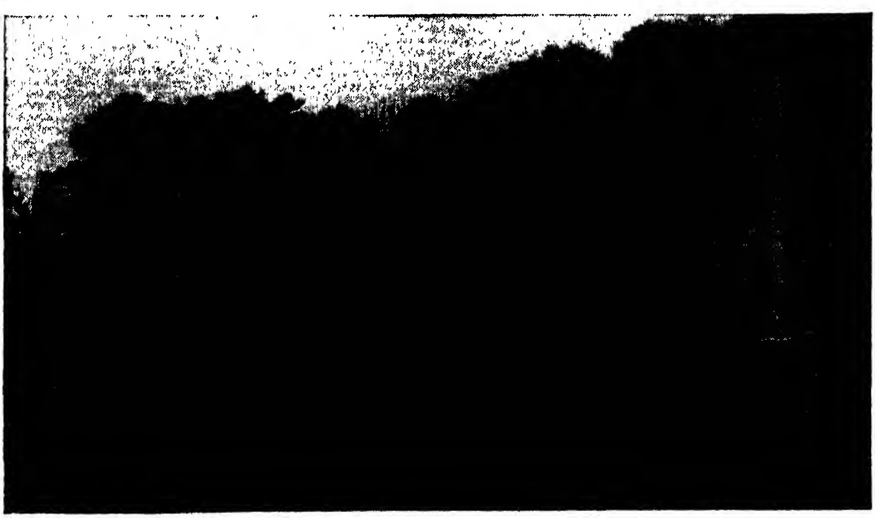


FIG. 14-15. North abutment of Pittsburgh Airport Parkway over Montour Run, Allegheny County, Pa. Notice the pilasters for the bridge bearings, the vertical joints, the horizontal grooving, and the weep holes. *(Courtesy of Parsons, Brinckerhoff, Hall, and Macdonald, Consulting Engineers, New York, N.Y., and Department of Public Works, Allegheny County, Pa.)*

on poor soils and on piles, the abutment will move as the earth supporting it moves. This was discussed in Art. 10-13.

6. Wingless abutments. A flanking-span abutment may become too long or too costly to be a practical structure. The end span of the superstructure, or a special short terminal span, may then be used. It may be

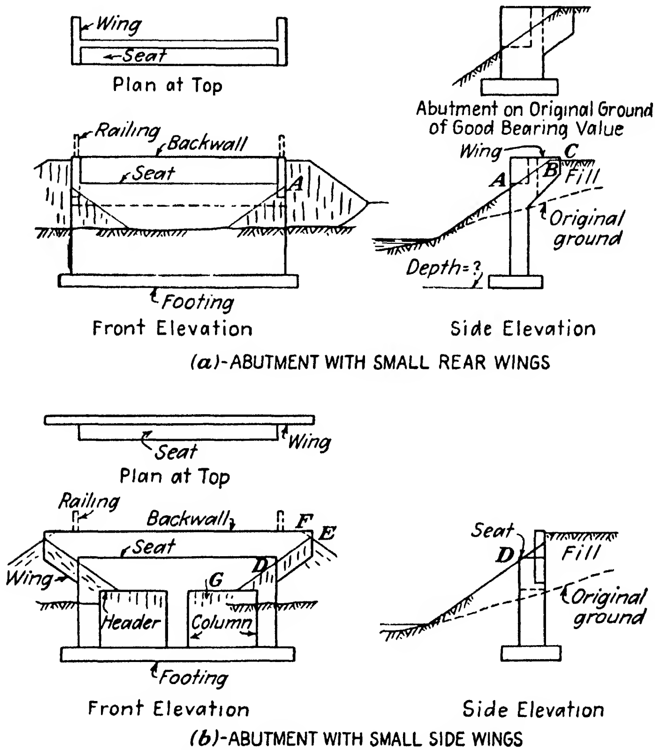


FIG. 14-16. 'Wingless" abutments.

supported upon a simple wall with a bridge seat, backwall, and end wall as shown in Fig. 14-16(*a*). This is merely supported upon a shelf excavated in the side of the cut and then surrounded by backfill. If soil conditions or the danger of scour require it, the wall may be deepened as necessary. When the roadway is on an embankment, two or more piers may be used to support the bearings directly or to hold a long bridge seat, and the backwall may span across them. The wings may be very small, and they may be perpendicular to the bridge seat or extended at the ends parallel to it to keep the fill from the bearings, an important matter.

Here there is no attempt to terminate the bridge by means of any massive appearing or special structure for aesthetic effect. The support is classed as an abutment only because it holds up the end of the superstructure.

If the end span of the bridge is a simply supported one, a shallow wall like that of Fig. 14-16(*a*) may be founded directly upon well-compacted embankment. If the latter settles a little, the abutment merely moves with it, and the span tilts slightly on its bearings. This is sometimes called a

FIG. 14-17. Inkster Road Bridge, Detroit Industrial Expressway. This is an excellent example of the use of side spans, sloping cuts, and small abutments at the tops of the slopes instead of massive abutments with heavy wing walls. Notice how simple and neat the structure appears. *(Courtesy of Michigan State Highway Department.)*

floating abutment. It may be advantageous where the soil is weak and some settlement is inevitable, but where this will not be noticeable or harmful. Of course, the wall should be strong enough to avoid breaking its back if the fill settles unevenly.

Figure 14-17 shows the use of what may be called a *wingless* abutment. Of course, such construction needs to be sketched in advance to make sure

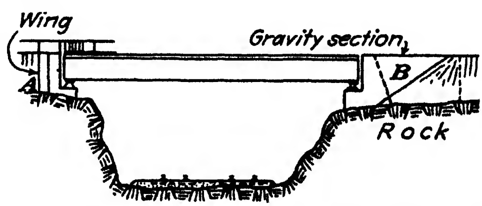
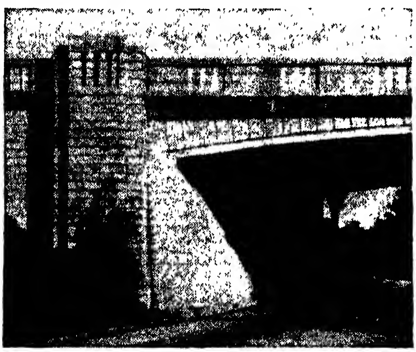


FIG. 14-18. Small abutments seated on bare rock alongside railroad cut.

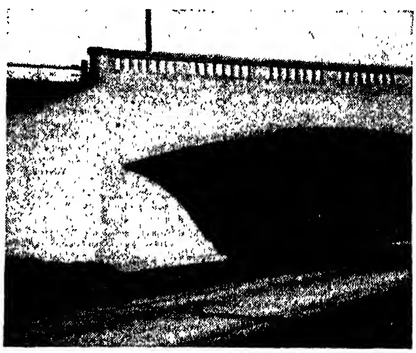
that the appearance will not be one of weakness and lack of finish. Also, it is more adaptable to overpasses and viaducts than to use at streams where floods may wash away the toe of the embankment.

There are situations in which abutments are so small that they may be of any or no special type. Such a case is pictured in Fig. 14-18. The rock alongside the cut is left bare, and the abutment is nothing much more than a bridge seat and a backwall. Small gravity-type construction may be all that is needed. On the other hand, much depends in this case upon the quality of the rock.

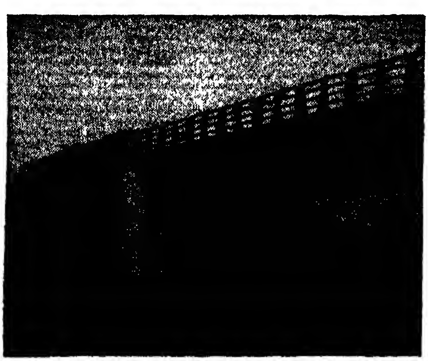
14-4. Abutments for rigid-frame bridges. Reinforced-concrete rigid-frame bridges are both attractive and economical for many conditions where short single or double spans are needed. The legs of the superstructure, whether barrel or ribbed construction, serve as the front wall of this abutment. This is shown in Fig. 14-20(a). Here the lateral pressure of the earth in the direction of the bridge is resisted by the superstructure itself; the abutment needs to hold the sides of the embankment only. Of course, there must be a footing to support the vertical load V and the hori-



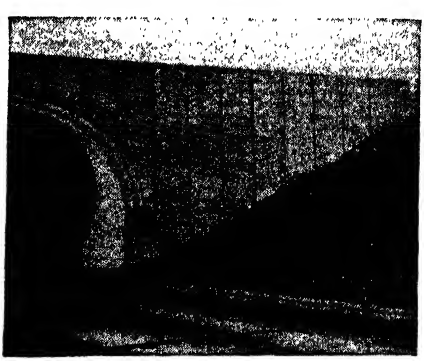
(a) A single-span, concrete, rigid-frame bridge with architectural treatment of surface, and with pylons and U-shaped abutment



(b) A two-span, concrete, rigid-frame bridge with curved, flaring wing walls at abutment



(c) A two-span, skewed, steel, rigid-frame bridge with simple, boxlike abutment



(d) A two-span, concrete, arch bridge with U-shaped abutment and vertical markings

FIG. 14-19. Some examples of highway bridges used to pass over the Merritt and Wilbur Cross Parkways in Connecticut. *(Courtesy of the Connecticut State Highway Department.)*

zontal thrust H of the frame. The latter naturally is opposed to the horizontal component of the earth pressure.

Since rigid frames are so dependent upon practically unyielding bearings, their use on weak soils should be questioned and studied carefully before being accepted. Vertical settlement of one end will cause a single-span rigid frame to tilt but not to fail; horizontal displacement of the abutment may have serious consequences. Variations in loads and temperature cause some changes in the magnitudes and directions of the bridge reactions. These should not cause harmful movements under any conditions. It is possible to tie the bearings together to resist the horizontal thrust, but this is not likely to be economical.

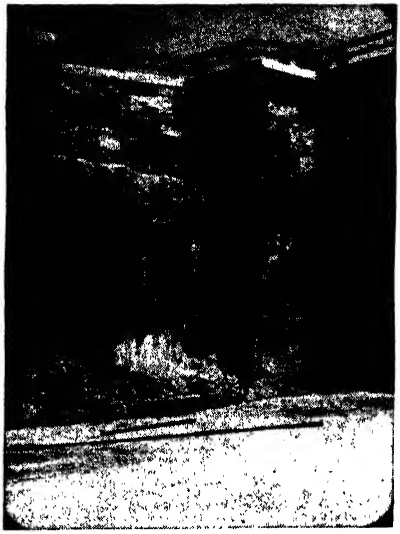


FIG. 14-19A. An attractive treatment for the abutment of a rigid-frame bridge over a roadway in a rock cut. *(Courtesy of the Port of New York Authority.)*

Figures 14-20(c) and (d) show to exaggerated scale the structural action of the rectangular pinned frame in (b) when acted upon by a live load and rise of temperature, respectively. Figures 14-20(e), (f), and (g) picture a simplified U-shaped abutment for such a bridge, assuming the upper roadway to be on an embankment. The leg is supported upon a hinged bearing along BB', Sketch (g), and it is free to deform between the fixed

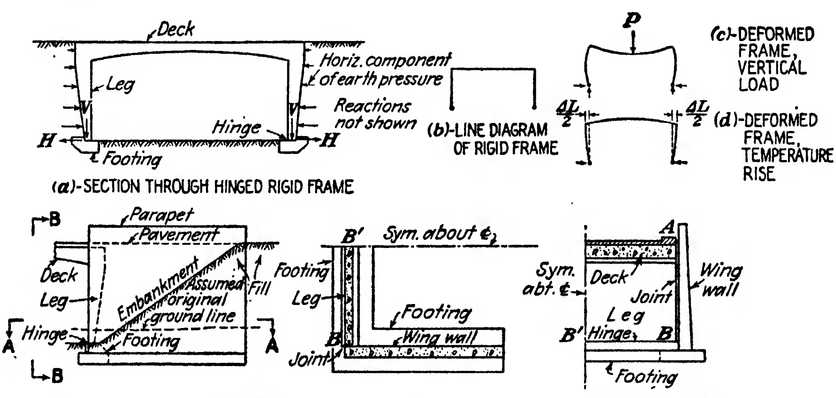


FIG. 14-20. Rigid-frame bridge and one type of abutment.

retaining-wall sides of the abutment. These walls are generally tied together by the footing under BB', but not elsewhere in this type. Joints AB should be flashed to prevent leakage.

This provision for movement of the legs of the frame is very important. Although the movements are small in angle and displacement, they are essential to the structural action. If the legs are keyed to or made a monolithic part of the abutment walls, something will break, or the structure will not act as intended. The result will be about as satisfactory as though a man were to try to walk with his knees in two pieces of cast-iron pipe or encased in concrete.

It might seem more simple to fix the bottom of the leg along line BB' in Fig. 14-20(g) than to hinge it. However, this section is generally thin

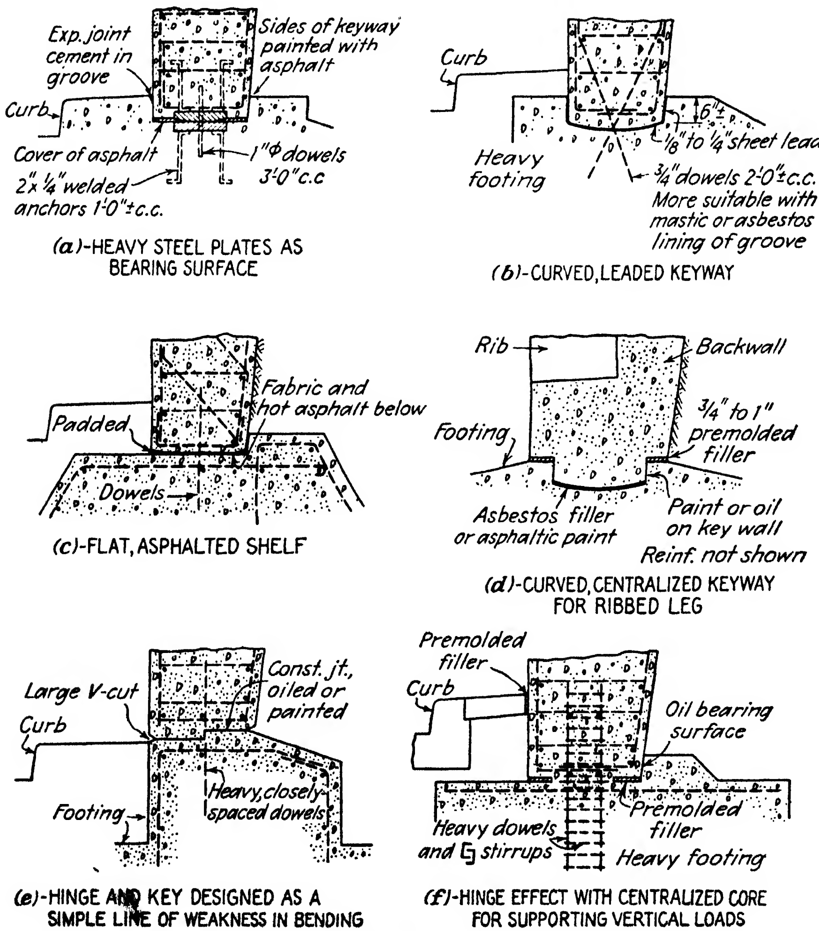


FIG. 14-21. Some arrangements for hinged bearings of concrete rigid-frame bridges.

because a certain amount of springing of the leg is essential, and since it is so narrow at BB', any restraint there will not have much relieving effect upon the bending movements in the main portion of the frame. A point of inflection will occur above BB' anyway. Any yielding of the footing or

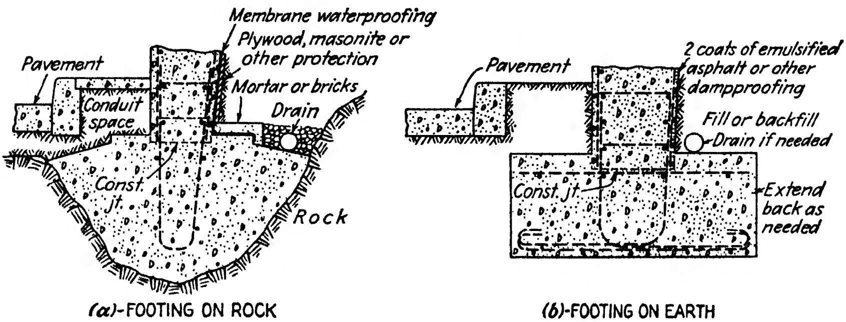


FIG. 14-22. Details of fixed-end footings for barrel-type concrete rigid-frame bridges.

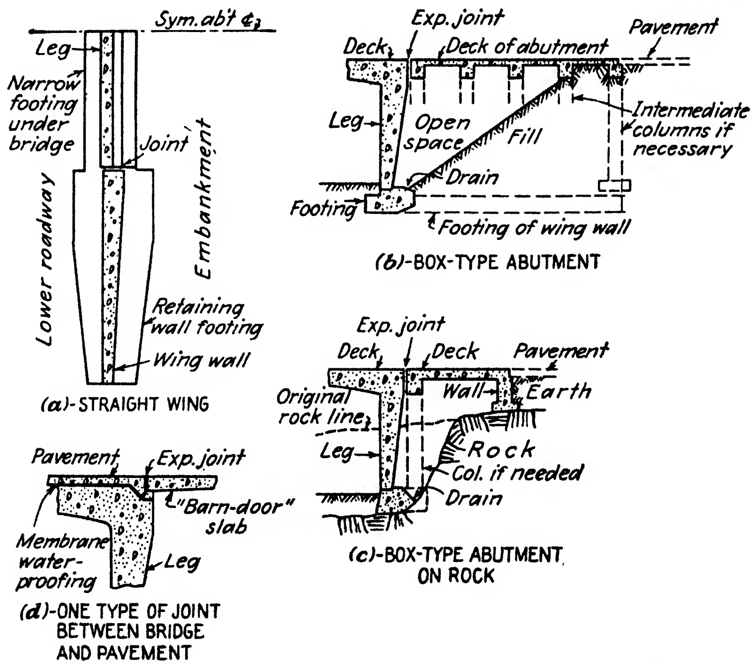
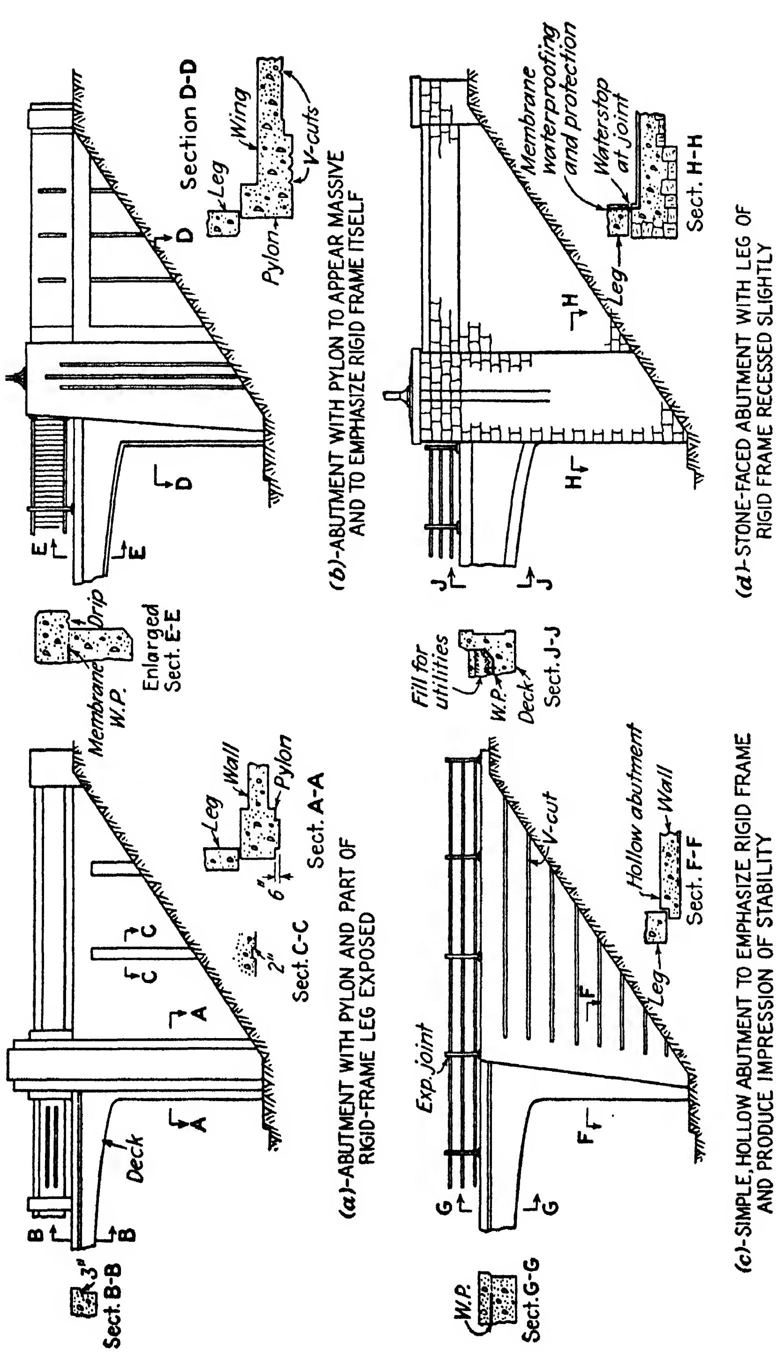


FIG. 14-23. Straight and box-type abutments for rigid-frame bridges.

of the materials in the leg will reduce the supposed restraining moment; it may even damage the structure locally and tend to produce a hinge where it was not intended to be. Unless the legs are unduly long so that more



(a)-ABUTMENT WITH PYLON AND PART OF RIGID-FRAME LEG EXPOSED

(b)-ABUTMENT WITH PYLON TO APPEAR MASSIVE AND TO EMPHASIZE RIGID FRAME ITSELF

(c)-SIMPLE, HOLLOW ABUTMENT TO EMPHASIZE RIGID FRAME AND PRODUCE IMPRESSION OF STABILITY

(d)-STONE-FACED ABUTMENT WITH LEG OF RIGID FRAME RECESSED SLIGHTLY

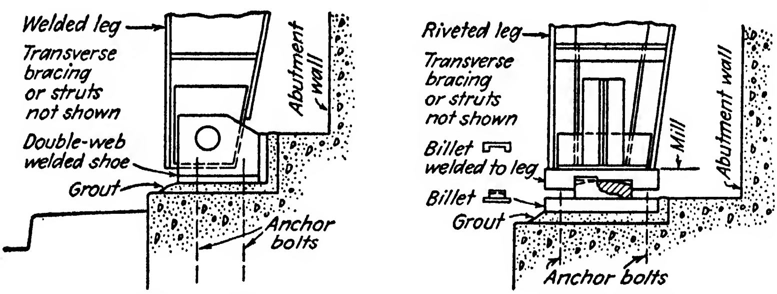
FIG. 14-24. Some studies of abutments of rigid-frame highway bridges to illustrate various architectural details.

stiffness is desired, it is generally advisable to design the structure with hinged supports. Some suggested details for hinged bearings are shown in Fig. 14-21, whereas some fixed ones are pictured in Fig. 14-22.

The details of abutments for rigid frames vary widely, depending upon the architectural style and treatment desired. When an underpass in a city is lined with retaining walls, the bridge may merely rest in a slot in the walls without any special abutments, as indicated in Fig. 14-23(*a*). When the upper roadway is on an embankment, it is possible to use sloping retaining walls parallel to the bridge legs somewhat as with straight-wing abutments. This, however, may not yield a pleasing appearance; the wings might better flare back at least 45°. Hollow box abutments can be used advantageously in some cases, thus eliminating large earth pressures against the leg and abutment. A few schemes for these are indicated in Figs. 14-23(*b*), (*c*), and (*d*).

Rigid-frame bridges are deservedly popular for use as single or double spans at grade-crossing eliminations. A few suggestions of various architectural treatment for the abutments of concrete bridges are shown in Fig. 14-24.

Longer rigid frames may be made of riveted or welded steel ribs with equalizing diaphragms between them, and with a concrete deck on top. It is possible to extend this deck down the rear of the legs as a curtain to hold



(*a*)-PINNED BEARING (*b*)-HINGED BEARING

FIG. 14-25. Two types of bearing for steel rigid-frame bridge.

back the earth from the steelwork. The motion of the steel, however, is likely to crack this cover noticeably unless it is well reinforced. Another and better method is to seat the bearings on a footing near the base of a retaining wall or curtain wall at the front of an abutment of wing, U, or box type. Naturally, this arrangement requires a more costly abutment than the former. A few suggestions for such construction are pictured in Fig. 14-25. They may be useful for the planning of abutments for other types of bridges.

14-5. Abutments for arches. The horizontal component of an arch reaction is generally large; with many ordinary arches it exceeds the vertical component considerably. Furthermore, arches are used for spans that are far greater than those of rigid frames. Hence, heavy loads and large thrusts are to be supported by the abutments. Therefore, good foundations are needed for this type of structure.

Rocky gorges are often ideal situations for arch bridges. The splendid structure shown in Fig. 14-26 crosses the Niagara River below the falls.

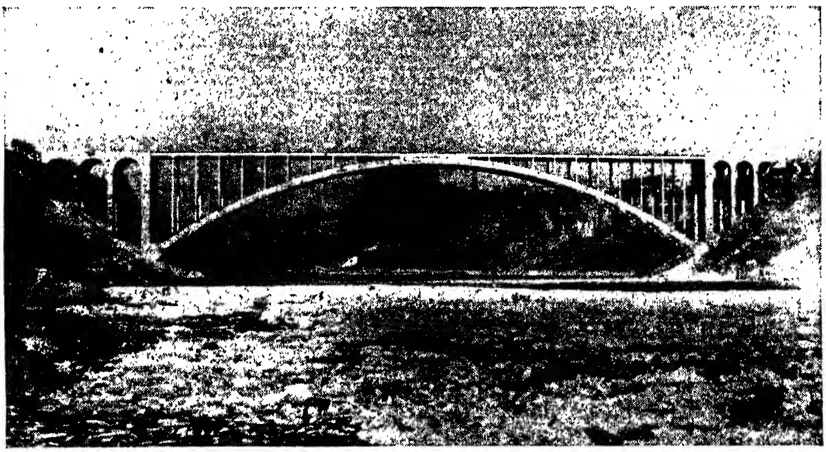


FIG. 14-26. The Rainbow Bridge over the Niagara River below the falls. This is a fixed-end steel arch founded upon rock. It was built for the Niagara Falls Bridge Commission, Niagara Falls, U.S.A.–Canada. Notice the ice in the foreground. *(Courtesy of Hardesty & Hanover, Consulting Engineers, New York, N.Y.)*

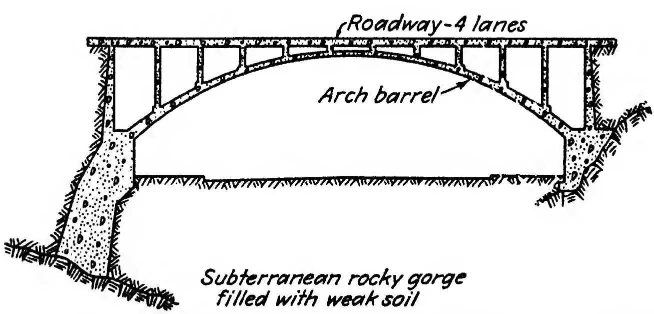


FIG. 14-27. A reinforced-concrete arch bridge supported upon rock at two different elevations.

The steeply sloping rock is strong and sound. The abutments, therefore, are practically nothing more than inclined footings that receive the thrusts of the bearings and transmit them to the bedrock. In this case, however, ice jams might be serious, hence the abutments are kept high enough above the water to avoid damage by the terrific ice conditions in spring floods.

Three-hinged arches are statically determinate structures. Therefore, slight yielding of the foundations may cause no serious harm. If the soil is weak and compressible, however, it may be difficult to determine whether or not the movement will continue until it becomes harmful. On general principles, such soils are not suitable for arches unless special provisions are made to prevent harmful settlements.

Of course, an arch may be desired on the basis of its attractive appearance. One such case is illustrated in Fig. 14-27. Rock was available at one end

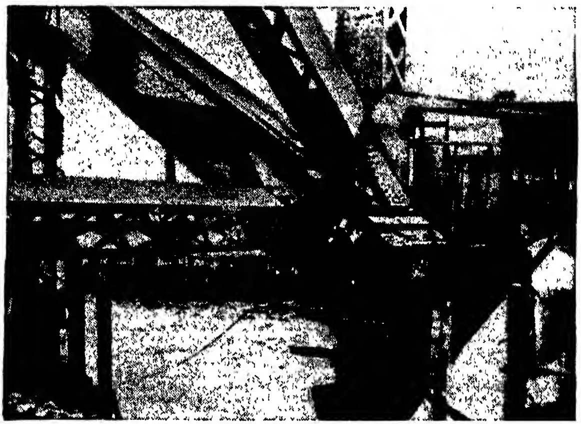


FIG. 14-28. Details at the end of the Bayonne Bridge. *(Courtesy of the Port of New York Authority.)*

but was some 20 ft. below ground at the other. Therefore, the latter was supported by a heavy extension or thrust block capable of resisting any combination of vertical loads and horizontal thrusts that might be applied by the arch. The wings of the abutment holding the fill at such a point as this should be supported upon piers or piles that reach the rock. Otherwise, differential settlement and cracking of the unequally supported parts may occur.

If an arch is designed with fixed ends, it may be difficult to make the abutments such that they will certainly offer the necessary restraint. Foundations on rock may be suitable; those on earth or piles may twist sufficiently to relieve a large part of the assumed restraint without anyone's realizing it unless the structure gets into difficulty. One might think that reinforcing rods or heavy tie rods could be grouted in drilled holes, thus fixing the abutment as strongly as needed. Some problems arise, however: the stretching of long rods under high tension, the possibility of eventual weakening produced by corrosion, inadequate grouting, weakness in bond or anchorage, and weakness in the rock caused by cracks and seams.

In Fig. 14-28 is shown one end of the Bayonne Bridge, a two-hinged arch. These bearings are designed to resist a thrust of something in the order of

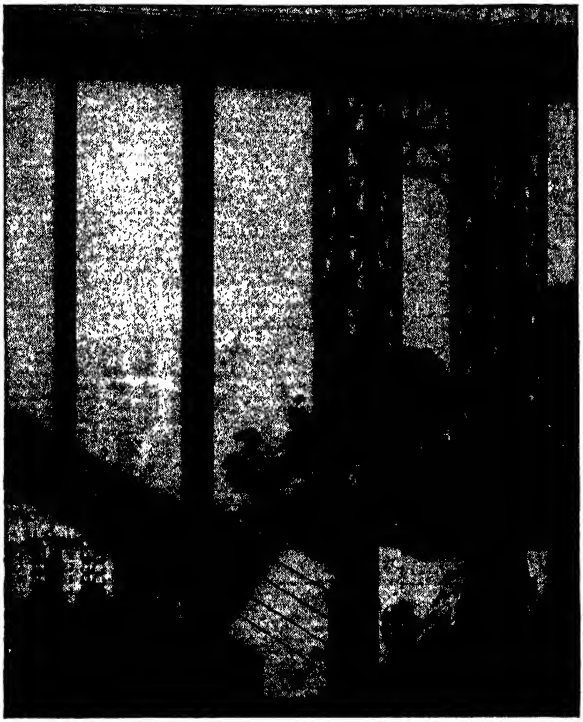


FIG. 14-29. Details of abutment and bearings of Henry Hudson Bridge at New York. Note the simplicity, neatness, and effectiveness of this rock-supported abutment. Notice also the use of steelwork instead of masonry as the terminus of the main span. *(Designed by D. B. Steinman, Consulting Engineer, New York, N.Y.)*

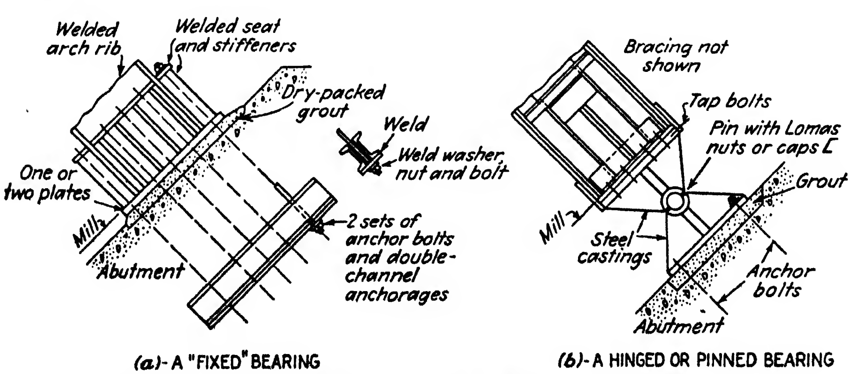


FIG. 14-30. Two arrangements for the bearings of steel-rib arch bridges.

30,000 kips. The details of the construction are shown in Fig. 8-8. Special attention should be given to the anchorage frames to support the structural shoes during grouting and when the erection forces were primarily vertical. Notice the eyebars, too, that were used to resist riverward forces caused by wind during erection when a portion of the trussing was supported by the

FIG. 14-31. View of a bearing, abutment, and approach piers of the Rainbow Bridge at Niagara Falls, N.Y. as seen when looking upward. Notice the details of the bearing, the fluting, and the rough, ridged surface of the massive piers. *(Courtesy of the Niagara Falls Bridge Commission, Niagara Falls, U.S.A.–Canada.)*

bearings and by one or more tall erection bents only. This illustrates the need for planning the scheme of erection when designing the abutments of a large arch.

Most large reinforced-concrete arches are supported upon fixed or restrained bearings that consist mostly of abutment blocks which the arch rib joins somewhat as would an inclined column. The reinforcing is carried into the abutment to provide the necessary resistance to bending. Bearings involving some of the principles shown in Fig. 14-21 may be used, but they are exceptional.

One type of fixed (restrained) and one hinged bearing for steel-rib arches are shown in Fig. 14-30.

14-6. Suspension bridge anchorages. The anchorages of suspension bridges differ considerably from ordinary bridge abutments. Their planning and detailed design for a major bridge usually require the work of several competent engineers with specialized knowledge and experience with such structures. However, the basic principles are relatively simple.

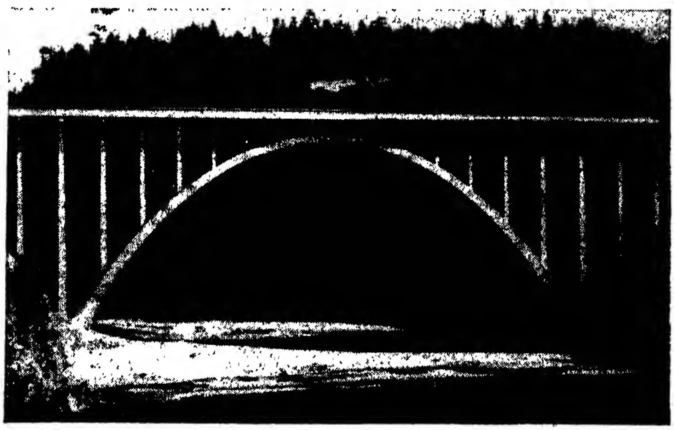


FIG. 14-32. The beautiful Russian Gulch Bridge in California. The arch rests upon a small abutment that is practically a flared spread footing that extends through the seamy shale near the surface to hard rock below it. *(Courtesy of the California Division of Highways.)*

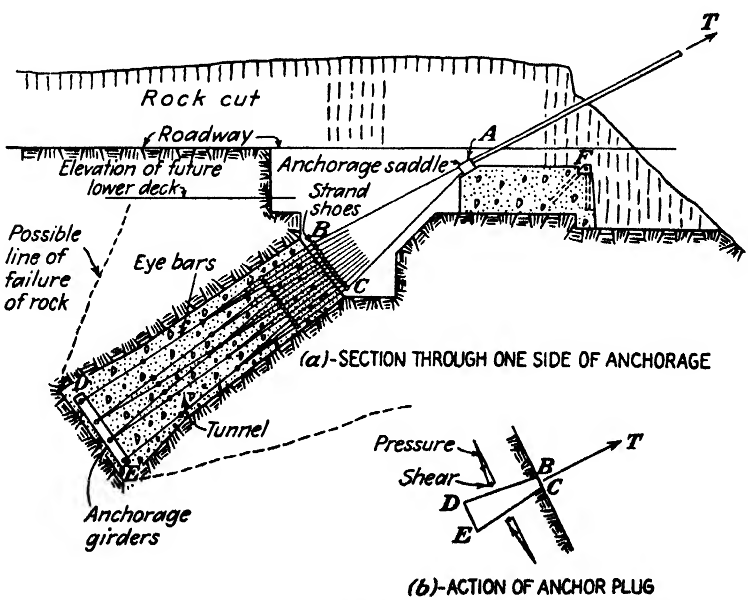


FIG. 14-33. Simplified section through New Jersey anchorage of the George Washington Bridge.

Figure 14-33(*a*) illustrates an anchorage built into a rock mass of practically infinite magnitude. This is almost an ideal situation. Figure 14-34(*a*) shows a "gravity" anchorage, this being a massive block of concrete that holds down the end of the cables. These two drawings represent the ordinary types of anchorage. Of course, variations and combinations may be made to suit special conditions.

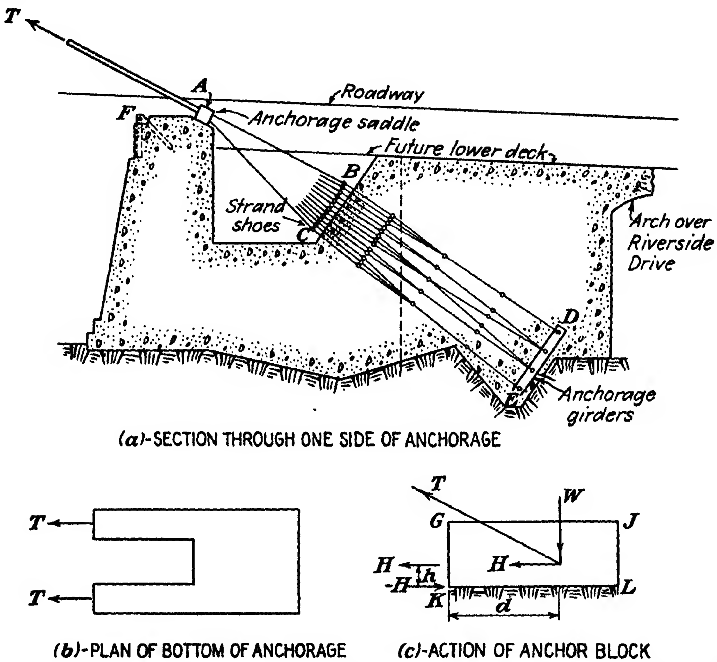


FIG. 14-34. Simplified section through New York anchorage of the George Washington Bridge.

Wire rope cables of small bridges may be fastened directly to anchorage masonry through sockets and structural ties. Parallel-wire cables of big bridges terminate in strand shoes outside the masonry. In Figs. 14-33 and 14-34, each 36-in. cable is built as sixty-one 4½-in. cables or strands. Riverward of the anchorage saddles *A*, these are squeezed together and wrapped to form the seemingly solid circular cables. Landward of these saddles, the strands are splayed vertically and horizontally so that each can be looped over a strand shoe. The saddles may be on rollers supported by masonry, or they may be on steel rocker bents. In any event, the saddles can move slightly with stress and thermal deformations of the cables so that the latter will not slide in the castings. The strands are bent downward more or less at the saddles so that they will not lift out under tension. Everything beyond the strand shoes is finally embedded in concrete.

The strand shoes are fastened by pinned connections to eyebars or fabricated heavy plates that transmit the cable pull back to anchorage girders near the rear of the masonry. These girders bear on the concrete and are designed to transmit to it a compression equal to the cable pull.

In Fig. 14-33(*a*), the portion *BDEC* is somewhat like the reverse of a cork in a bottle. As shown in (*b*), the cable tries to pull the concrete plug out of the rock. The harder the cable pulls, the tighter is the plug jammed against the surrounding mass. This causes pressures around the concrete that tend to burst the rock, or to tear out a conical portion of it in front of *DE*. Of course, careful exploration is made in advance to assure that such failure of the rock cannot occur, then the anchorage simply cannot get away. The safety factor may be indeterminate, but the anchorage is designed to be undoubtedly reliable.

In Fig. 14-34, over 100,000 cu. yd. of concrete constitute the U-shaped block into which the anchorage steelwork extends. The cables try to tip this block about some point near its front edge. The moment Wd must exceed the overturning moment Hh by at least the desired safety factor. The resistance to sliding should be equally safe. In this anchorage, the rock is cut to offer a mechanical shear lock. This sliding problem may be very serious in the case of anchorages founded upon earth.

Such structures deserve careful study because a bridge worth many millions of dollars depends upon their safety.

14-7. Miscellaneous details. The details of the joint between the end of a superstructure and an abutment are important; they may be rather troublesome also. Many arrangements are possible, some being illustrated in Fig. 14-35. Not only should the joint be made so that its parts are structurally strong enough to support concentrated loads and impact, it should be so designed that it will not let through the tire of a bicycle, a pedestrian's heel, or anything that may cause an accident. It should also be made so that drainage water will not flow over and stain the abutment, thus spoiling its appearance. This last may be difficult to avoid, but it should be accomplished when attractiveness is desired.

Fills placed behind abutments for the support of traffic will almost inevitably settle somewhat. It may be advisable to use a bituminous pavement on the fill from the backwall of the abutment to a point 50 or 100 ft. away. When the settlement causes a depression (and bump to traffic) next to the backwall, this temporary pavement may be built up or otherwise repaired easily. When compaction has ceased some years later, a permanent pavement may be built.

A better remedy is the use of a "barn-door" slab that can bridge from the backwall to the fill far enough away so that the angular rotation of the slab will cause an acceptable ramping effect only. This construction is illustrated in Fig. 14-36.

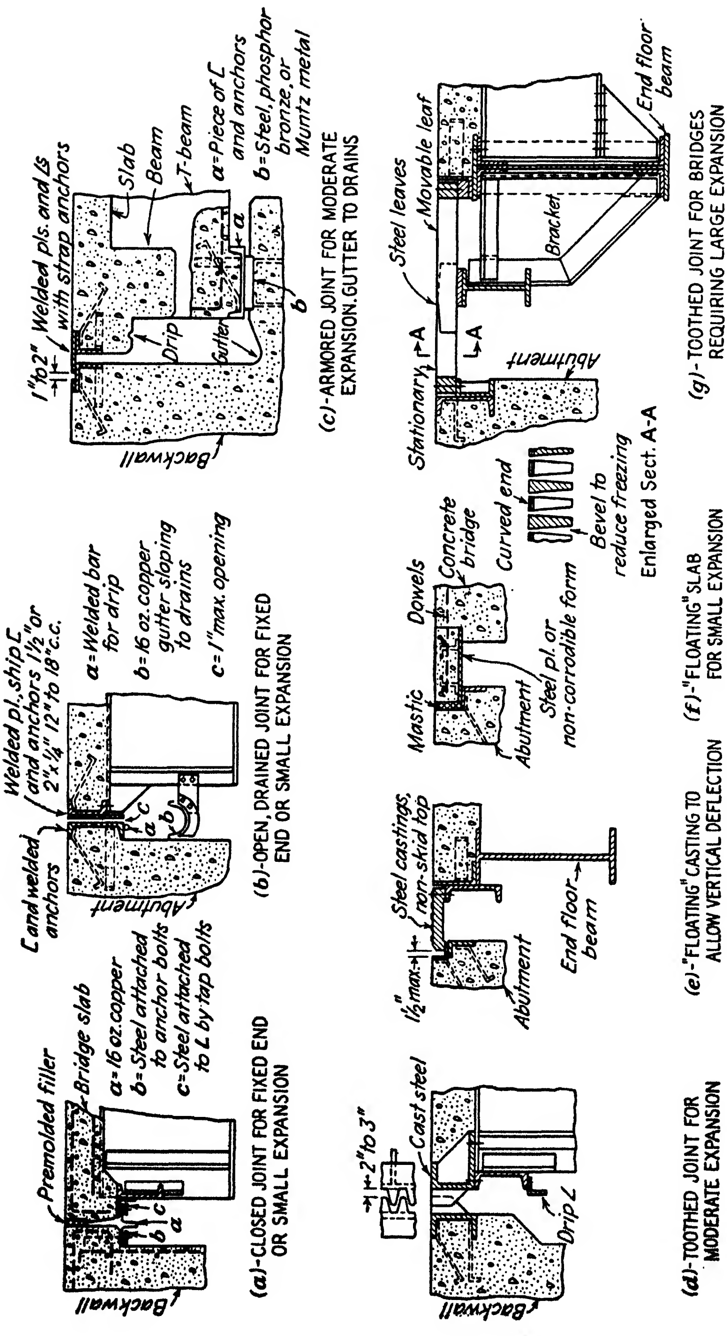
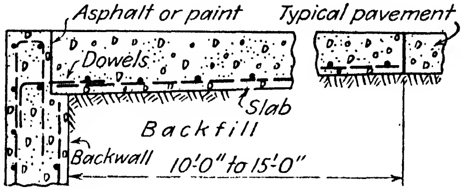
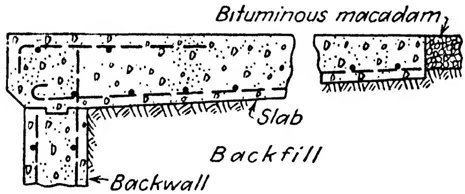


FIG. 14-35. Some details of expansion joints at bridge abutments.

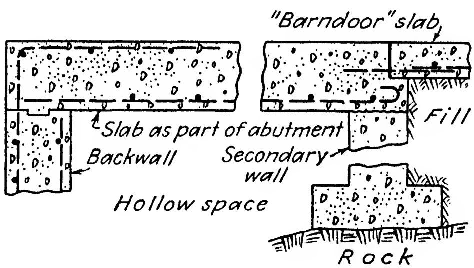
Adequate waterproofing of abutments may seem costly but it is important when durability and appearance are to be at a maximum. It is usually sufficient to use a two-ply membrane with adequate protection, and to have waterstops at the contraction joints. Drainage of the space or fill back of an abutment should always be as adequate as circumstances permit.



(a) SEAT, SLAB DOWELLED TO BACKWALL



(b)-SLAB EXTENDED ACROSS BACKWALL



(c)-HOLLOW ABUTMENT AND AUXILIARY SLAB

FIG. 14-36. "Barndoor" slabs spanning from abutments to ground or fill.

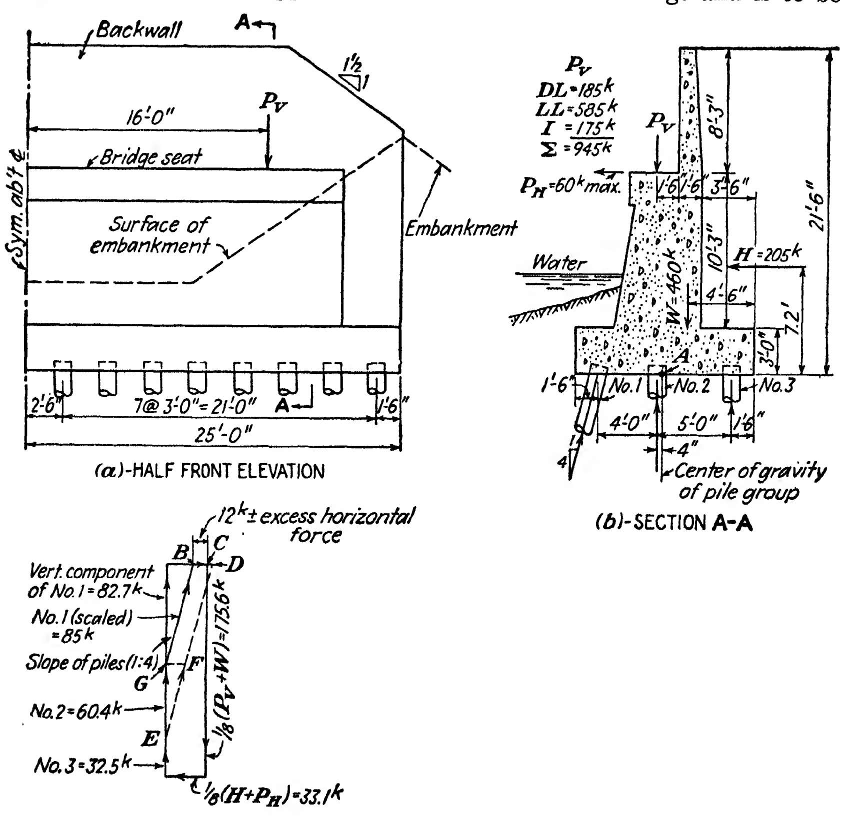
Weep holes alone may become clogged with dirt or closed temporarily by icicle action.

Should an abutment be battered? A slope of ¼ to ½ in. per ft. is sometimes desirable for the front and wings. In other instances, as in Fig. 14-24, a greater batter may be used to create a particular architectural effect. Some architects have stated that an extensive high wall that is absolutely vertical may appear to lean forward.

In building an abutment, as with other concrete structures, care should be used to obtain a dense durable concrete and to have a surface finish that produces a pleasing effect. The use of V cuts or straight-edge finishing at construction and contraction joints will generally avoid ragged unsightly junctions of pours. The entire design should be planned to facilitate construction, provide adequate strength, and yet produce a structure that is attractive and serves its purpose well.

14-8. Problems in analysis. Assuming that the desired size, shape, dimensions, and details of an abutment have been selected and that a sketch has been made of it, there remains the question of the strength and safety of the structure. Computations should then be made to prove that the abutment will be satisfactory. Such calculations will vary considerably, depending upon each particular problem. Some structures may require careful investigation. Others may be so obviously safe that few computations need be made, most of the designing being based upon approximations that are determined largely by the engineer's judgment.

Example 1. A rather massive and simple abutment is pictured in Fig. 14-37. It is to support a double-track railroad bridge and is to be



(a)-HALF FRONT ELEVATION

(b)-SECTION A-A

(c)-FORCE DIAGRAM FOR ONE ROW OF PILES

FIG. 14-37. Analysis of pile reactions for an abutment of a two-track railroad bridge.

founded upon piles. For the loads shown, compute the forces upon the piles. The passive resistance of the earth in front of the abutment may be neglected, although its weight above the footing is included in W. Assume that the maximum allowable load per pile is to be 40 tons.

This abutment is designed as a unit. The structure is to distribute the bridge reactions over the group of piles as a beam or stiff block. Because of symmetry, one-half of the abutment may be analyzed by itself. The forces shown in Fig. 14-37 are given on this basis, allowing for buoyancy below the water line and using 120 p.c.f. for the unit weight of compacted fill in air. The procedure is as follows:

1. Compute the moment of inertia of the piles about their center of gravity, treating each pile as a unit, as in Art. 10-6:

$$I = 8(4.33^2 + 0.33^2 + 4.67^2) = 325 \text{ pile-ft.}^2$$

2. Compute the overturning moment and eccentricity about the center of gravity of the pile group:

$$M = 205 \times 7.2 - 460 \times 1.67 + 945 \times 0.33 + 60 \times 13.25 = 1{,}810 \text{ ft.-kips}$$

$$c = \frac{1{,}810}{460 + 945} = 1.29 \text{ ft., left of point } A\text{, Sketch } (b)$$

3. Compute the vertical forces on each of the piles, using $p = \dfrac{V}{N} + \dfrac{Mc}{I}$:

$$\text{No. } 1 = \frac{1{,}405}{24} + \frac{1{,}810 \times 4.33}{325} = 82.7 \text{ kips}$$

$$\text{No. } 2 = \frac{1{,}405}{24} + \frac{1{,}810 \times 0.33}{325} = 60.4 \text{ kips}$$

$$\text{No. } 3 = \frac{1{,}405}{24} - \frac{1{,}810 \times 4.67}{325} = 32.5 \text{ kips}$$

These results show at once that pile No. 1 will be somewhat overloaded under these conditions.

4. Draw the graphical force diagram in Fig. 14-37(c) for one row of piles and for $\frac{1}{8}(P_H + H)$. This shows that an excess horizontal force of 12 kips must be withstood by each group of three piles, by the abutting resistance of the earth in front of the abutment, or by friction. The second of these resistances may be worth investigating if there is no danger of erosion of the portion of the embankment that projects around the front of the abutment. It is safer, however, to make the piles capable of resisting this excess. Therefore, pile No. 2 might be battered also. As shown by EF and FD, the two front piles will then be capable of causing equilibrium.

Besides making pile No. 2 a batter pile, should the excess of 5 kips on pile No. 1 be accepted? A more conservative design is preferable. This pile might be moved about 1 ft. farther to the left in Fig. 14-37(b), all of the piles might be shifted to the left with respect to the footing in order to reduce the eccentricity, or the group might be rearranged or increased to have four piles in a row under the same or a wider footing. The reader should experiment with these revisions and determine for himself what is the best solution. The designs of reinforcement for the footing, backwall, main section, and end of the wing are ordinary routine matters.

Example 2. Figure 14-38 shows a plan for a U-shaped abutment to support a highway overpass composed of a series of parallel steel girders

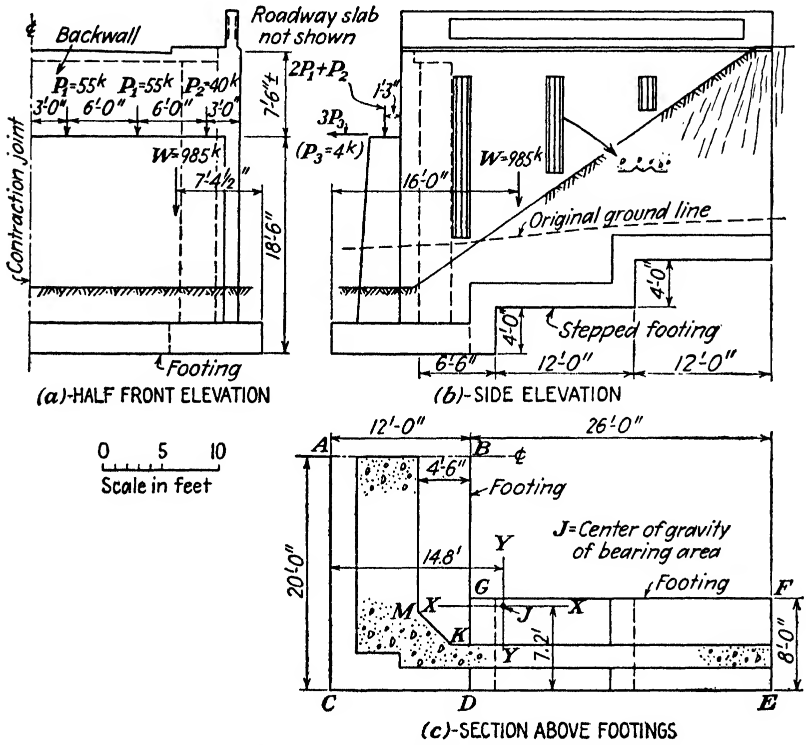


FIG. 14-38. Analysis of a U-shaped abutment.

holding a roadway slab at their tops. The general arrangement has been made on the basis of practical construction and the desired appearance. The dimensions have been chosen arbitrarily as a starting point. Is the structure satisfactory, assuming the soil to have an allowable bearing value of 6 k.s.f.?

This problem is one of those practical cases where no exact and accurate analysis can be made. In fact, it is not necessary to make one if some reasonably approximate methods can be used to prove that the structure will be safe and not unduly costly for the type of structure and for the appearance desired. Different engineers may have various methods of attacking such problems. The author himself might use certain methods on this problem but other methods in different circumstances. In practice, an engineer will invent his own procedures to accomplish what he thinks is necessary and desirable.

It may seem strange that such a problem is given here. However, it is done deliberately to show the reader that there are problems in foundation engineering which are not simple and which cannot be answered by substitution in a few formulas. In such cases the problems are solved by methods not usually admitted or advertised outside the office. Yet when built the structures are safe and practical.

First, test the design to see if the bearing on the soil is satisfactory for the load conditions shown. Since there is a contraction joint at the center of the abutment, the portion illustrated will tend to tip diagonally toward the corner C, Fig. 14-38(c) because of the lateral earth pressure inside. The greatest bearing on the soil will probably occur at this point.

The following procedures are used with the knowledge that the results will probably be well on the side of safety.

1. General assumptions:

a. Compacted fill and embankment weigh 120 p.c.f.

b. Dimensions not given will be scaled from the drawing.

c. Angle of internal friction ϕ of soil $= 30°$.

d. Horizontal active earth pressure $= wh\left(\dfrac{1 - \sin\phi}{1 + \sin\phi}\right)$.

e. The passive resistance of the soil outside of the abutment will be neglected, an assumption that is far on the side of safety.

f. A surcharge equal to 2 ft. of earth will be applied over the soil within the abutment wings.

g. The footings of the steps will be assumed to be in one plane—that of the footing under the front wall. This will be done as though the abutment were sliced vertically along the offsets of the wing footings, each slice (including the earth) being slid downward until the footings line up. Admittedly, this assumption is a broad one.

2. Weight of abutment:

a. The approximated weight of concrete and of the earth over the footings is found by summing up the parts and is recorded in Fig. 14-38(b).

b. The center of gravity of the abutment is also computed and recorded in Sketches (a) and (b).

3. Properties of bearing area:

a. The center of gravity is computed and recorded in Sketch (c), and the area is 448 ft.^{2}

b. The moments of inertia are computed,

$$I_{xx} = 13{,}100 \text{ ft.}^4$$
$$I_{yy} = 54{,}900 \text{ ft.}^4$$

c. Using the method explained in Art. 8-7, the angle θ is determined and the axis $X'X'$ located.

$$I_{xy} = 20 \times 38 \times 4.2 \times 2.8 - 12 \times 26 \times 10.2 \times 6.8 = -12{,}700 \text{ ft.}^4$$

$$\tan 2\theta = \frac{2(-12{,}700)}{54{,}900 - 13{,}100} = -0.607$$

$$2\theta = 148°40', \quad \sin 2\theta = 0.52, \quad \cos 2\theta = -0.854$$

$$\theta = 74°20', \quad \sin \theta = 0.963, \quad \cos \theta = 0.27$$

The footing is redrawn in Fig. 14-39 to make the construction clearer.

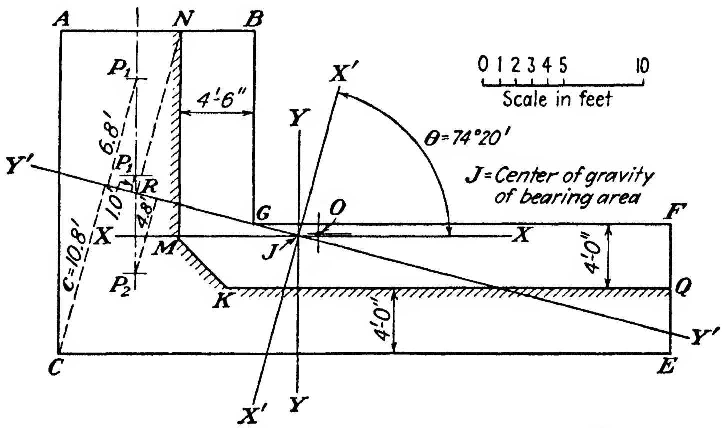


FIG. 14-39. Graphical construction for part of computations for soil pressure under footing for Fig. 14-38.

4. The minimum I about $Y'Y'$ of Fig. 14-39 is

$$\frac{I_{xx} + I_{yy}}{2} - \frac{I_{xx} - I_{yy}}{2} (\cos 2\theta) + I_{xy} \sin 2\theta$$

$$I_{\min} = 34{,}000 - 17{,}900 - 6{,}600 = 9{,}500 \text{ ft.}^4$$

5. The overturning moments normal to $Y'Y'$ are now to be estimated. The footing as redrawn in Fig. 14-39 should be referred to. The line $NMKQ$ represents the inside of the concrete walls. With the 2-ft. surcharge, the maximum depth of earth is 28 ft. The value of $\frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1}{3}$. The dimensions are taken from Fig. 14-38.

a. Earth pressure on front wall, assuming a triangular pressure diagram 28 ft. high:

$$H_F = \frac{120 \times 28 \times 14 \times 16}{3} = 251{,}000 \text{ lb. (approx)}$$

The longitudinal overturning moment is

$$M_F = 251 \times 9.33 = 2{,}340 \text{ ft.-kips}$$

The component normal to $Y'Y'$ is

$$M_F \cos\theta = 2{,}340 \times 0.27 = 632 \text{ ft.-kips}$$

b. Earth pressure on side wall (approx), using the assumed triangular pressure diagram applied to slices based upon the various portions above the stepped footing:

$$\text{Front portion } H_1 = \frac{120 \times 28 \times 14 \times 6.5}{3} = 102{,}000 \text{ lb.}$$

$$\text{Middle portion } H_2 = \frac{120 \times 24 \times 12 \times 12}{3} = 138{,}000 \text{ lb.}$$

$$\text{Back portion } H_3 = \frac{120 \times 20 \times 10 \times 12}{3} = 96{,}000 \text{ lb.}$$

The sideward moment of these portions about the bottoms of their footings is

$$\Sigma M_s = 102 \times 9.33 + 138 \times 8 + 96 \times 6.67 = 2{,}700 \text{ ft.-kips}$$

$$\Sigma M_s \sin\theta = 2{,}700 \times 0.963 = 2{,}600 \text{ ft.-kips}$$

Notice that the axis of moments is approximated as a line sloping up through the centers of the bottoms of the respective footings, or that all are pulled down to the level of the front footing. It does not seem to be necessary to use lever arms for all the sections of the wing wall as though they rotated about the plane of the bottom of the front footing. This is because the ground under the stepped footing is supposed to be trustworthy. This is done in spite of the fact that the bearing area of Fig. 14-39 is used as a plane at this lower level. If the reader will be less confused, he may assume that the wing wall is cut into slices and that the two rear ones—including the earth as well—are skidded downward 4 and 8 ft., respectively.

c. Lateral loads P_3:

$$M = 3 \times 4 \times 18.5 \times 0.27 = 60 \text{ ft.-kips}$$

d. Live loads P_1 and P_2, using scaled lever arms from $Y'Y'$ of Fig. 14-39:

$$\Sigma M = 4.8P_2 - 1.0P_1 - 6.8P_1 = 4.8 \times 40 - 7.8 \times 55 = -237 \text{ ft.-kips}$$

e. Dead load of abutment: As scaled from Fig. 14-39, the center of gravity O of the abutment lies about 0.4 ft. back of axis $Y'Y'$. Therefore,

$$M_{DL} = -985 \times 0.4 = -394 \text{ ft.-kips}$$

f. Total overturning moment about $Y'Y'$, Fig. 14-39, is the sum of the components normal to the axis:

$$\Sigma M = 632 + 2{,}600 + 60 - 237 - 394 = 2{,}660 \text{ ft.-kips}$$

6. Maximum toe pressure at C:

$$\Sigma W \text{ for vertical loads} = 985 + 2 \times 55 + 40 = 1{,}135 \text{ kips}$$

Assuming that there is pressure under the entire area of the base, and treating the latter as a plane surface at the level of the front footing,

$$P_c = \frac{\Sigma W}{A} + \frac{(\Sigma M)c}{I_{y'y'}} = \frac{1{,}135}{448} + \frac{2{,}660 \times 10.8}{9{,}500} = 5.5 \text{ k.s.f.}$$

The result of the preceding calculations is somewhat less than the 6 k.s.f. allowed. Admittedly the calculations are approximations at best, but they give some scale to the problem of bearing at the corner of the footing. Most of the overturning moment is caused by the earth pressure on the wing wall. One might now estimate the back pressure and counteracting moment caused by the one-quarter cone of embankment outside the wing. This will decrease the computed toe pressure. However, its effectiveness will not be great, and there would probably have to be too much movement if the abutting power of this cone were to be developed.

Furthermore, there appears (by comparison of lever arms and the results of item 6) to be an uplift at the rear corner B, Fig. 14-39. Technically, since tension is impossible under the footing, the preceding formula for P_c should be replaced by some method that considers triangular rather than trapezoidal pressure diagrams. This may be both difficult and unnecessary. It is desirable to make the structure so that the tendency for uplift does not occur, or so that the safety of the structure is not endangered in any case. If the back pressure of the embankment is estimated, and if it is not large enough to reduce the overturning moment sufficiently, the rear of the footing BG might be moved back to increase the weight of earth over this part of the structure or the toe of the wing wall might be broadened.

A checkup of the danger of sliding shows the following:

$$\Sigma H \text{ normal to } Y'Y' = 251 \times 0.27 + (102 + 138 + 96) \times 0.963 + 12 \times 0.27 = 394 \text{ kips}$$

Required $f = 394/985 = 0.4$, which seems to be safe for a reasonably good soil. Furthermore, the computations have made no allowance for the resistance of the ground outside the abutment, which will be considerable.

If the abutment is accepted as it is, or with a widening of the front footings, the reinforcement can be computed for footings and walls by using the results of the preceding computations. However, special care should be given to the connection between the front wall and the wing at the fillet MK.

The reader should notice that much of the difficulty with this abutment comes from the presence of the contraction joint at the center, Fig. 14-38(a). If the front wall were solid, the two wings would tend to counteract each other. There would then be less difficulty with overturning, but shrinkage and cold weather might cause the abutment to crack as explained previously. Furthermore, this problem shows the advantages of the box and flanking-span types of abutment for conditions where they are suitable.

Example 3. Assume the soil profile and the other data given in Fig. 14-40. What type of abutment is the most desirable for this double-track railroad bridge?

Notice that it is over 32 ft. from the ground at the front of the bridge seat to the base of rail. The clay is weak, its frictional resistance is not depend-

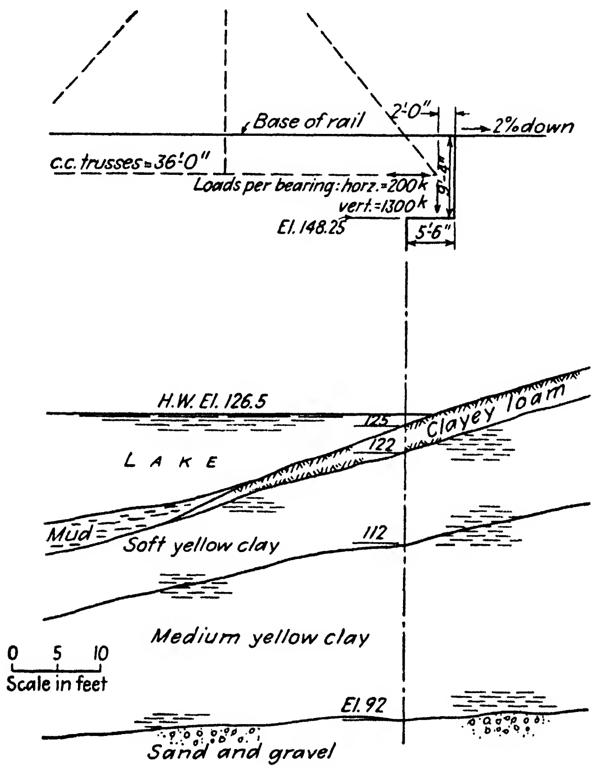


FIG. 14-40. Profile at site of abutment for a railroad bridge.

able, and its compaction may cause considerable settlement. Therefore, a straight-wing or beveled-wing abutment would be objectionable because of the great weight and overturning. Even if piles are used, such a severe overturning is inadvisable, and the weight of the fill will be likely to cause movement of the clay. The same comments apply to a U-shaped abutment.

A box-type abutment would be better but the deck would be long and broad, and it would have to be very strong to hold train loads. From the standpoint of appearance, the big wing walls would probably look unduly massive.

Then a flanking-span abutment might be considered. However, the deck would be so heavy that it would become practically a short bridge.

Therefore, it seems that the following construction may be economical and practicable:

1. Use a tapered concrete shaft about 7 or 8 ft. square at the top under each truss bearing.

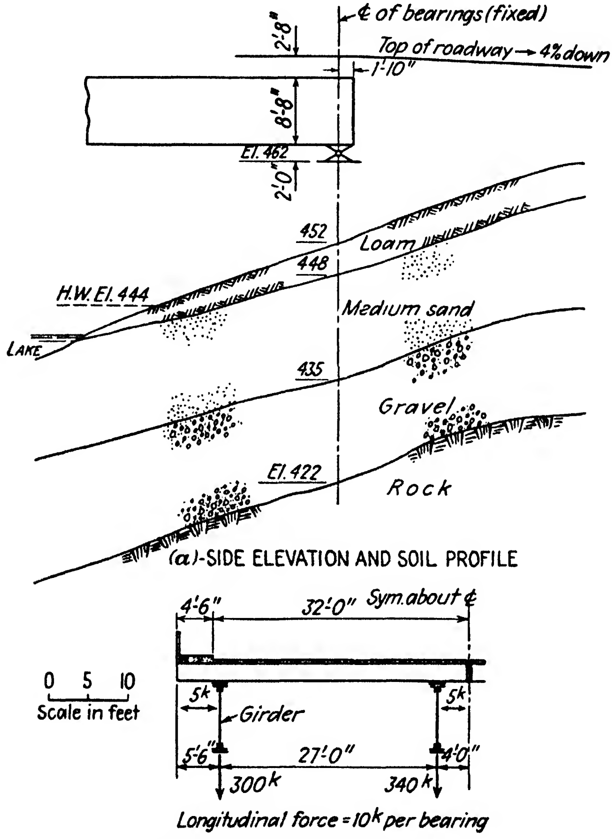
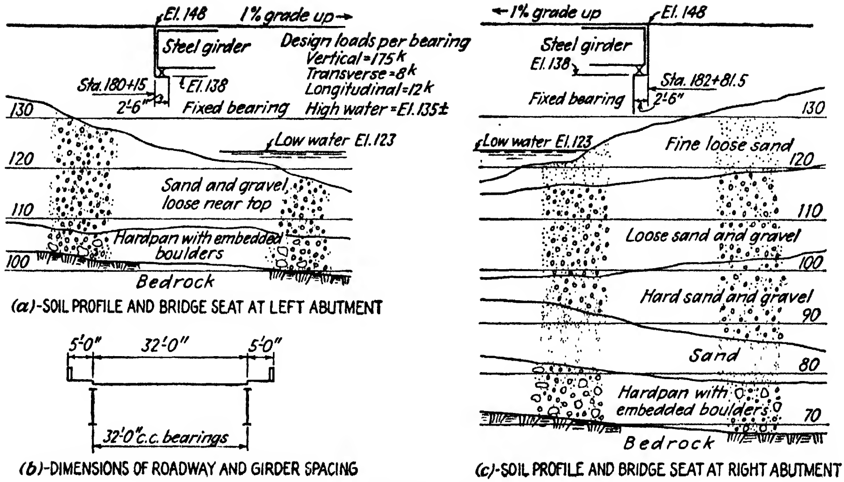


FIG. 14-41. Problem in design of an abutment for a highway bridge.

2. Place these shafts on a heavy rectangular footing with the bottom at approximately El. 112.

3. Support the footings on cast-in-place or precast concrete piles driven into the sand and gravel, the piles being designed to resist the vertical loads and the overturning caused by braking and traction. The work can be carried on inside small cofferdams.

4. Use two single-track deck plate-girder bridges to span from the end of the truss bridge to a shallow simple beveled wing abutment some 40 or 50 ft. shoreward, supported on concrete piles if necessary.

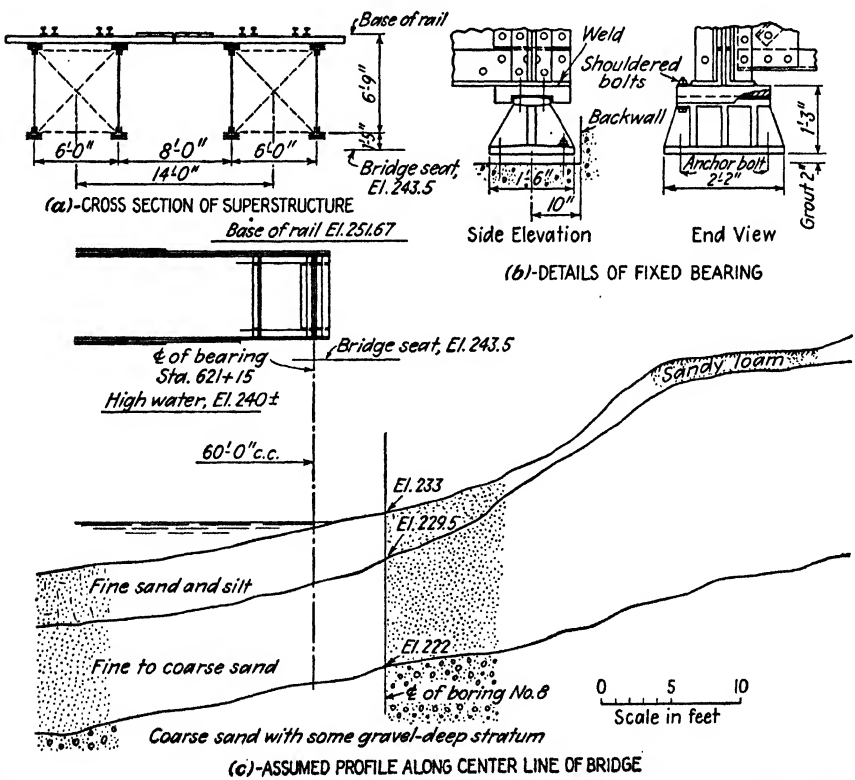


(a)-SOIL PROFILE AND BRIDGE SEAT AT LEFT ABUTMENT

(b)-DIMENSIONS OF ROADWAY AND GIRDER SPACING

(c)-SOIL PROFILE AND BRIDGE SEAT AT RIGHT ABUTMENT

FIG. 14-42. Conditions at the abutments of a proposed bridge.



(a)-CROSS SECTION OF SUPERSTRUCTURE

(b)-DETAILS OF FIXED BEARING

(c)-ASSUMED PROFILE ALONG CENTER LINE OF BRIDGE

FIG. 14-43. Data showing superstructure and soil conditions at site of proposed abutment of a railroad bridge.

5. Connect the tops of the concrete shafts by means of a reinforced-concrete portal strut approximately 8 ft. deep that will be wide and strong enough to support the plate girders and the end stringers of the truss bridge. Widen the piers if necessary to suit the girder bearings and the strut under them.

Thus this structure turns out to be a pier rather than a real abutment.

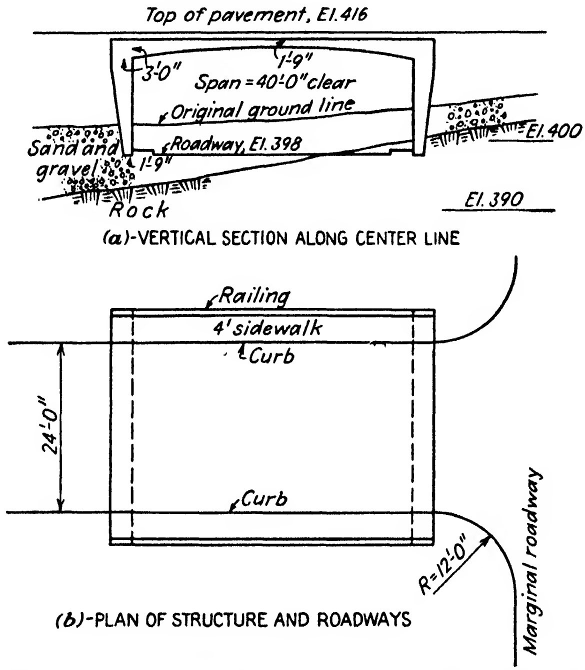


FIG. 14-44. Problem in planning abutments and foundations for a rigid-frame, concrete-barrel highway overpass.

PROBLEMS

14-1. Design an abutment for a bridge to meet the conditions shown in Fig. 14-41. This highway bridge consists of two symmetrical portions with a longitudinal joint separating them. The bridge is to cross the narrow portion of a long shallow crooked fresh-water lake in a mild climate.

Suggestions: Floods and ice are not important here. This is a six-lane boulevard supported by four girders spaced as shown. Assume the allowable bearing pressures from Table 4-5. Use concrete construction.

14-2. Figure 14-42 pictures the conditions at both abutments of a three-lane deck-girder bridge, over a river approximately 250 ft. wide with moderate current at flood stage. Design the abutments for this structure.

14-3. Figure 14-43 shows the general construction and the bearings for a double-track deck-girder railroad bridge. The end span is 60 ft. long. Design an abut-

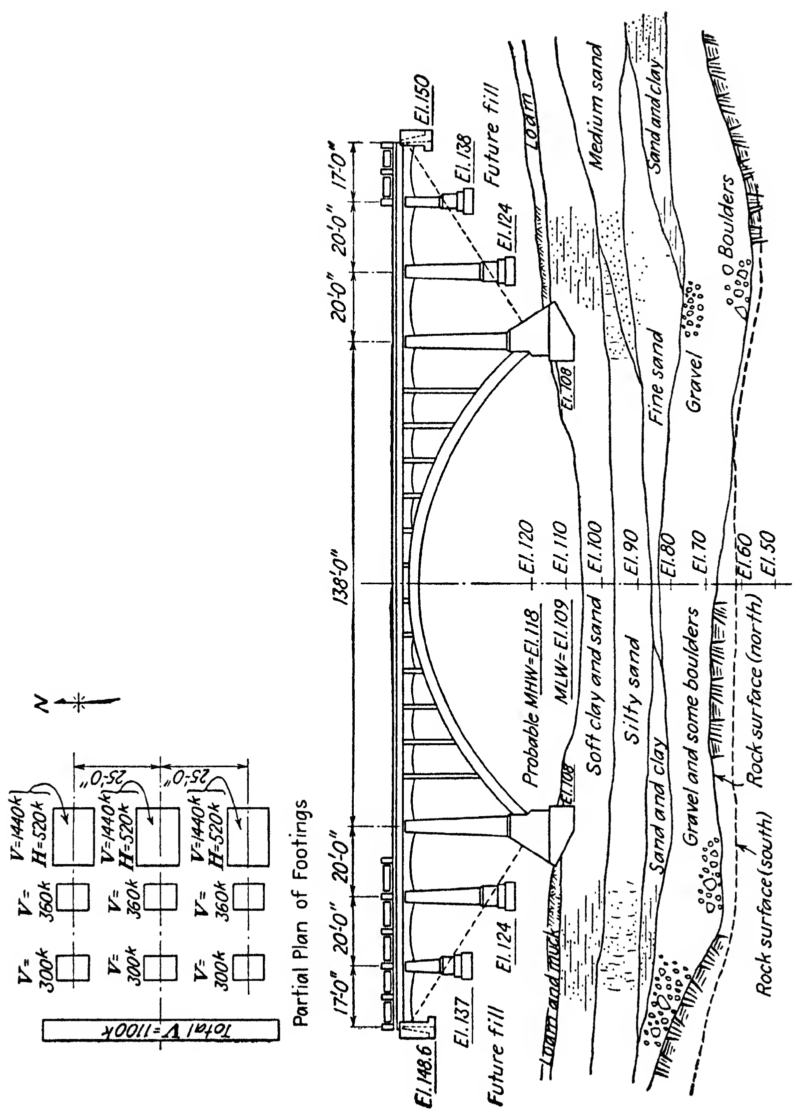


FIG. 14-45. Soil profile at the site of a proposed concrete arch highway bridge.

ment for this bridge. For preliminary purposes, use the load data in Figs. 13-34 to 13-38.

14-4. Design the abutments for the highway bridge shown in Fig. 14-44. They are to be built of concrete and are to have pylons at the corners for architectural purposes and to hold light standards. Assume hinged bearings, with a vertical load of 16 kips and a horizontal load of 4 kips per lin. ft. applied at these bearings. The railing is to be pipe posts on a 6-in. curbing and two pipe rails with welded vertical spindles between them. Invent two abutments that are architecturally attractive and harmonious even though perhaps different because of the marginal roadway at the right.

14-5. Figure 14-45 shows a three-ribbed reinforced-concrete arch bridge to be built across a small river near the Atlantic Coast of southern New England. The architect wants this type of structure. The loads are given in the plan view. Design the foundations for the structure.

15

UNDERPINNING

15-1. Introduction. The word *underpinning* is used in foundation engineering to denote the process of transferring the weight of an existing structure from its original foundation to temporary supports, and from them to a new permanent substructure. It may also denote the construction of new foundations under the structure without the intermediate use of temporary supports. When the work is completed, the structure is said to be *underpinned.* This may be necessitated occasionally by the inadequacy of the original foundations, or by their harmful settlement. In most cases, however, it is caused by the fact that new work in the immediate vicinity of the structure, or even in the building itself, requires new foundations at a lower level.

Some of the modifications and construction likely to make underpinning of existing structures necessary are the following:

1. Provision of a basement in a building that had none before, or of a subbasement where only an ordinary basement existed previously.
2. Installation of tunnels, sewers, water pipes, electrical ducts, and air ducts under or near columns and walls. This is likely to occur when alterations are being made in an industrial plant.
3. Construction of pits, tanks, sumps, wells, and elevator pits.
4. Lowering of basement floors.
5. Construction of subways, and lowering of adjacent streets for grade-crossing eliminations and other street improvements.
6. Lowering of the watertable by adjacent excavations or pumping that endangers the safety of footings on wooden piles.
7. Increase of loads on columns or walls that requires replacement of the present foundations. A revision that merely adds to the size and strength of a footing to permit greater loads without taking the original load off the footing is not underpinning in a strict sense.
8. The construction of deep foundations in positions that endanger present ones.

9. Lowering of the ground surface by regrading so that footings have to be set down farther to prevent damage from frost action.

10. Scour, or the threat of dangerous scouring, caused by flowing water.

15-2. General principles. The conduct of underpinning work is likely to require the exercise of great ingenuity and engineering judgment. Conditions usually seem to be special for each job. The soils, the water conditions, the character of the new work, the details of the old structure, the maintenance of use of the structure during the conduct of the work, the magnitude of the loads—all these vary greatly.

It is easy to say that the work should be done so as to support the existing structure safely, and to avoid damage to it. This is the essence of the problem, and the problem is a tough one. The work will almost always be slow and expensive. As illustrated later, the underpinning has to be done a little at a time and usually involves a great deal of hand labor. Many times the work has to be done with whatever materials are at hand or readily procurable.

Generally, the load has to be taken off the existing footing and held by temporary means while a new support is provided. The load then has to be transferred to the new bearing. It is important to do all this in a manner that does not require or produce raising or lowering of the structure sufficiently to cause cracking or other harm to it. Adjustment to proper elevation is necessary as well as strength of supports.

One means for obtaining adjustment is the use of wedges. The amount of their allowable movement is generally small unless the wedges are driven, shims or blocks inserted to hold the load, the wedges taken out, and the wedges redriven on top of additional shims. Large forces can be raised or supported by wedges, but the force exerted is not easy to control. For example, an enthusiastic workman told to drive in the wedges with a sledge may hit them so hard that he lifts the structure too much, and therefore cracks it. A lazy man may do the opposite, with the result that the structure settles and cracks when its original support is removed. Double wedges are preferable to single ones. Any wedges should be arranged so that they cannot slip out or be knocked out accidentally.

Another method of adjustment is the use of jacks. Screw jacks are good and will hold their load indefinitely if used properly, but a single one has very limited capacity. Several may be used in a battery, but great care is necessary in operating them so that the load is distributed satisfactorily to each one in the group. Hydraulic jacks may easily have an individual capacity of 100 to 200 tons. However, the loads should not be left on them for long periods because of possible leakage or breaks in piping. It is best to use the jacks to hold or raise the load, then block up the structure with shims and wedges. Later on, the wedges can be driven out or the jacks can again take the load so that the shims can be removed and the load transferred to the new supports. Both types of jack give a fairly good control over the force exerted by them.

Turnbuckles, bolts, and threaded rods are useful in obtaining adjustment when the loads are suspended from some overhead structure or framing.

Ground water is often troublesome in underpinning operations. Sumps and pumps may have to be used, or it may be necessary to install wellpoints. Underpinning operations in wet runny soil are especially dangerous.

When work requires underpinning of structures that are the property of others (or even work near these structures), the owner of the new work should examine these structures carefully, take extensive photographs showing their present condition, and make a report upon the subject, having the report reviewed and approved by someone who is competent to do so. He is then in a position to defend himself against unjust claims for damages that are allegedly caused by his construction operations. It is a reflection upon human nature that, after such operations, the cracks in a building suddenly become prominent and important. Photographs showing their previous existence are very helpful. In fact, a contractor is wise if he takes pictures and makes a report on his own account if the owner does not do so. The contractor can then protect himself if necessary.

It is obvious that underpinning work should be done with extreme care. Nevertheless, here is the story of one actual case:

A tunnel was being built for a new conveyor under the first floor of an industrial plant. The building was unencased steel framing with several floors or heavily constructed platforms. It housed and supported equipment for roasting (drying) ore. The tunnel was to pass close to one of the interior columns. The excavation had proceeded past this particular column which, with its old footing attached, was blocked up by heavy vertical timbers under the concrete. Blasting was required for the excavation, and it had been conducted carefully. One day, a workman apparently thought that, "if a little dynamite is good, more will be better." He therefore put in much heavier charges than usual, without the knowledge of anyone else.

When the blast was set off, it knocked all the shores out from under the column footing. The column settled about 3 in. but did not fall down. Seemingly, the beams and roof framing acted as a set of cables, cantilevers, and whatnot that transmitted the load to surrounding portions of the structure. The timbers were hastily wedged back under the footing temporarily.

The next day, the men tried unsuccessfully to jack the column back to proper elevation. They then collected all the jacks that they could find around the plant. These were placed under the footing. The only result was that several of them broke without budging the column. The distortion of the building could not be eliminated. Some tracks that were supported by the floor near this column were shimmed up later and the column footing was underpinned with concrete. The building is still in service, but such operations are not recommended.

15-3. Underpinning columns. Although the procedures used are similar in many respects for underpinning columns and for underpinning walls, the former will be considered first because it involves supporting a single isolated load.

Assume a steel-framed factory building with columns supported upon isolated spread footings. A small basement is to be built under the first floor. This requires underpinning for one of the interior columns.

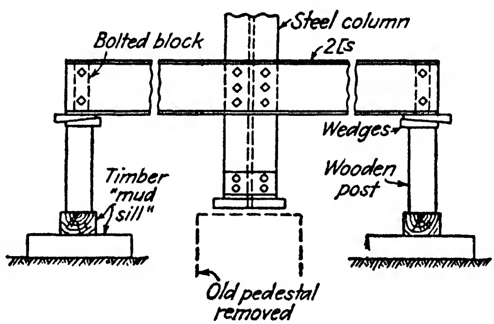


FIG. 15-1. One method of shoring steel column.

In this case, the dead load of the structure is relatively light. As shown in Fig. 15-1, it is feasible to drill a few holes in the column flanges and to bolt two heavy channels thereto. These can be supported upon timber posts and grillages called *mud sills,* with wedges (or jacks) under the channels at the top of the posts as shown, or under the posts. The load can be transferred from the footing to the channels by driving the wedges sufficiently. The supporting system should be braced carefully to make sure that no disturbance of it occurs during the work of demolishing the old footing, excavating a shaft for the new footing that is to be below the basement floor, and building the new substructure. When the new concrete is set, the base of the column can be grouted or dry-packed, then the wedges can be loosened so that the load of the column is transferred to the new pier and footing. It is generally inadvisable to try to pour the concrete clear up to the billet because adequate and even bearing may not be obtained.

Many variations of the general scheme illustrated here may be used. Wedges or jacks at one end of the temporary beams may be sufficient. If a small initial settlement of the structure is not harmful, the wedging might be done under the column base plate after the new pier is ready for grouting. However, this procedure is not advisable. It is better to pick up the load first to make sure that there will be no weakness in the temporary supports or harmful settlement of the mud sills before the old foundation is removed.

Another method that may be used for steel-framed structures, and some times for reinforced-concrete ones, is illustrated in Fig. 15-2. Inclined posts supported upon mud sills are wedged under beam or truss connections to form an A-frame that will support the column until the new foundation is ready. This may be dangerous if the beam connections are inadequate.

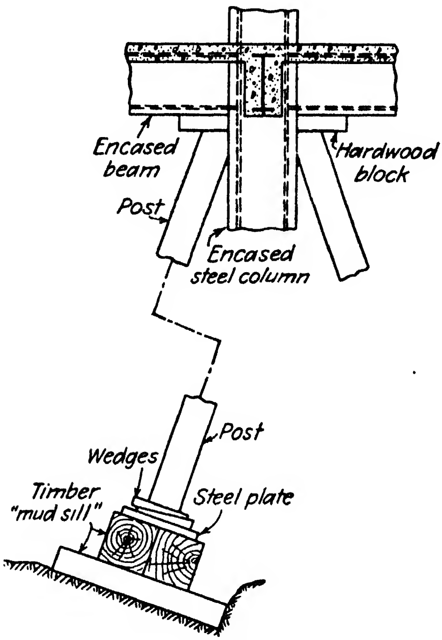


FIG. 15-2. Wedged, inclined posts under beams.

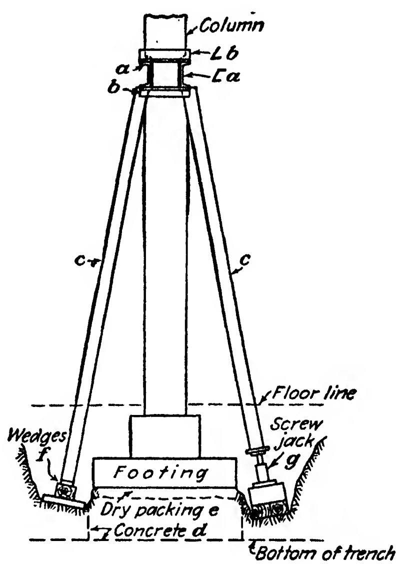


FIG. 15-3. Inclined shoring of a column.

In the case of concrete beams, the reversal of the end shear and its large magnitude may be especially serious.

One method of underpinning a concrete column is shown in Fig. 15-3. A trench is to pass next to the footing, and the latter is to have a concrete pad placed below it to reach to the level of the invert of the trench. It is easy to make connections to structural steel, but it is difficult in the case of reinforced-concrete columns. In this case, two chases are gouged out of the concrete as far as the reinforcement. Two steel channels a are erected as a yoke and tied together by bolted angles b. They are then grouted in place to bear on the concrete. The operation may then be conducted as explained for Fig. 15-1, or an A-frame may be used as shown here with posts c on wedges or jacks. The best procedure is to pick up the load on the A-frame, excavate under half of the footing, pour half of the concrete pad *d*, and then dry-pack between this new concrete and the footing. When this is done, repeat the process for the other half, then release the load onto the footing, dismantle the shoring, and patch up the column. However, the column will be permanently weakened by the portion that is chipped out because plaster patches will remedy the appearance only. In general, such a procedure is dangerous.

One abutment (or two-column pier at the end of a main truss) of the Summit Bridge near Chesapeake City, Md., settled and tilted when heavy rains washed out part of the bank. A serious slide threatened. The Army Engineers saved the structure by doing the following:

1. The approach structure was dismantled and some 20,000 cu. yd. of earth removed from behind the abutment in order to relieve the lateral pressure.
2. A concrete deadman was buried 50 ft. back of the abutment. This was tied to the bottom of the substructure to help prevent sliding riverward.
3. A wellpoint system was installed to a depth of 20 ft. to remove the ground water, thus preventing lubrication of the soil.
4. Eight steel H piles were driven alongside each of the two shafts.
5. Slots were cut in each pier shaft, as indicated in Fig 15-4. Beams 36 in. deep were then erected on top of the piles and concreted to bear. Long rods between the beams were tightened to hold them together. This steelwork formed a pile-supported grillage under each column so that they could support the structure if the soil under the footing yielded further.
6. The bridge was then raised by jacks placed on the tops of the columns, concrete pedestals were poured, and the bearings were reset.

What may be called underpinning of a column is illustrated in Fig. 15-5. The soil is dry or moist sand. The floor alongside the row of columns is to be lowered several feet to make a large shallow pit and a retaining wall is

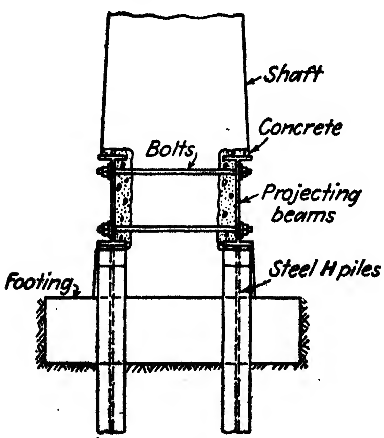


FIG. 15-4. Use of permanent yoke.

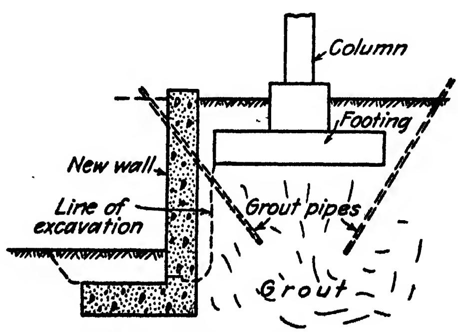


FIG. 15-5. Use of grout as underpinning.

to support the earth. Under and in the vicinity of the footings the ground is first grouted. Pipes are driven down in various positions as indicated, and the cement grout is forced in slowly under high pressure until a sufficient mass of earth is thought to be converted into a sort of very weak mortar. When this is set thoroughly, the excavation of a trench for the wall footing is made, using shoring where it seems necessary. This excavation is done by careful picking of any grouted soil that must be removed. The wall is then built. Below the footing, the concrete of the wall might well be poured directly against the ground to ensure support for the latter. If this is not done, sand or gravel should be packed into the intervening space. The rest of the excavation can be made at will.

In the remodeling of an industrial building there was an interesting example of underpinning with cement grout and of the inadequacy of hearsay and assumed information. This structure had been built on fairly level ground and approximately 75 ft. from the foot of a steep outcropping of rock. New heavy equipment was to be installed in the structure, and the engineer inquired about the adequacy of the soils. The former operators said, "The building is all founded on rock. Can't you see it right there?"

There was already in the basement a heavy slab that formerly supported some small compressors. Taking the information given by the operators as correct, the engineer built on the basement slab a heavy pedestal for a large compressor. It consisted of two solid concrete walls and a thick cap. The old slab was to constitute the footing.

When the compressor was started, it acted somewhat like a rocking horse. It had to be shut down immediately. Investigations then showed that rock was about 100 ft. below the building and that the ground below the thin sandy top stratum was a fine silty sand of poor bearing power. What was to be done? After considerable study, the engineer decided to grout the soil to a depth of about 30 ft. under and around the machine in order to solidify a large volume of ground and to form a sort of enlarged footing. This was done by using a pressure of 100 p.s.i., by starting at the deeper level, and by grouting so as to form something like horizontal layers. After the job was completed, the machine operated with negligible vibration.

15-4. Underpinning of walls. Figure 15-6 illustrates one method of underpinning brick, masonry, and concrete walls, or concrete foundation walls that support intermittent exterior columns. This shows a case where a vehicular subway or underpass is to be built close to and alongside an old building.

The general method is the excavation of pits some 3 or 4 ft. wide at intervals of 15 to 30 ft. This allows the load to be resisted temporarily by the intervening undisturbed soil. It illustrates the utilization of the safety factor in the bearing capacity used in the original design. Thus, pits represented by A_1 and B_1 are dug, using sheathing generally to make sure that the intervening soil does not slip. Then a solid concrete pier (or a footing and a thin pier) is made up to about 2 or 3 in. below the old wall or footing. After the concrete has set properly, steel or hardwood wedges are driven between the new pier and the old structure, as shown, in order to force load into the pier. The rest of the space is then packed with stiff mortar.

The next step is the excavation of pits next to the first ones, as shown by A_2 and B_2. The adjacent concrete piers are built as before. Then

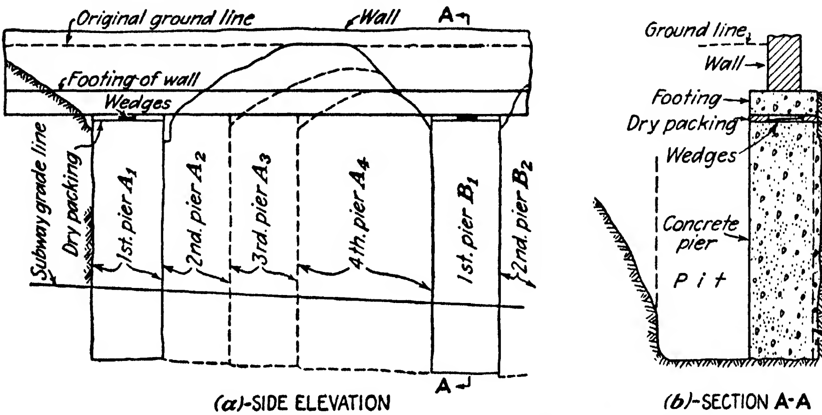


FIG. 15-6. Underpinning a continuous wall alongside a new subway.

follow other pits and piers. The last ones, pictured by A_4 in Fig. 15-6, are finally excavated and concreted, thus forming what amounts to a continuous foundation wall. It is seen that no attempt is made to lift the old structure, unless the wedging does so unintentionally.

In the case of a concrete foundation wall supporting light columns 20 ft. c.c., it may be most advantageous to construct the piers under the columns first, making them perhaps 4 ft. wide so that the bearing area will be considerable. A series of middle piers may be constructed next to hold the central portions of the wall. Then the remaining spaces can be excavated and filled in.

The use of needle beams is shown in Fig. 15-7. The inside of the building is assumed to be accessible for work. A series of holes some 6 to 10 ft. or more apart is made in the wall. Steel beams are placed through them and supported upon mud sills and blocking of some sort. The spaces between the wall and the beams are filled with concrete or mortar to obtain strong bearing. The beams may then be wedged up to transfer the load of the wall to them, or the wall may be allowed to settle until the beams support it (a dangerous procedure). The excavation, concreting, wedging, and packing can then be done more or less in the open and in quantity. The needle beams and their supports can be removed later, and the wall can be patched up.

A variation of this procedure is shown in Fig. 15-8(*a*). Here the needle beams are placed under the footing. Part of the basement floor is removed.

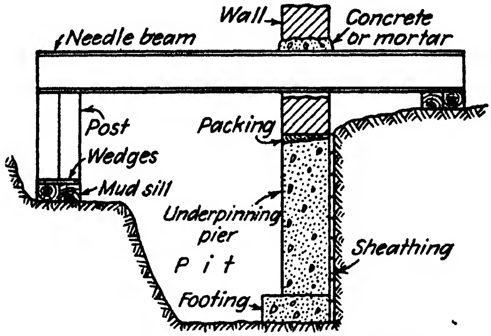


FIG. 15-7. Illustration of use of needle beam.

The general excavation outside is carried to some such line as *EFGH*. That inside is made along line *ABCD*. Slots along *JK* are dug out next at intervals, the needle beams *f* are slid through and placed on their supports *b* and *c*. The outer ends of the beams are then jacked to pick up the load,

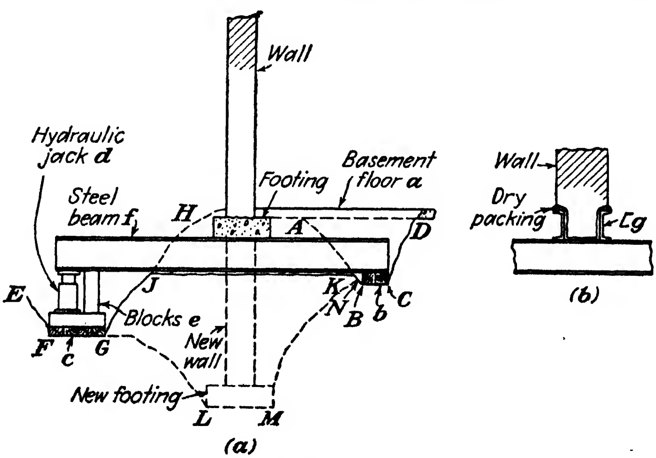


FIG. 15-8. Needle beams under footing of a wall.

and blocks *e* are put in to support the beams. The excavation *GLMN* is then made, the concrete footing and wall are poured, any wedging and packing are completed, the shoring is dismantled, and the job is finished.

A footing is shown purposely in Fig. 15-8(*a*). On account of the appearance, it should not be left exposed as shown. If it has a good formed outer edge, the new wall may be made thick enough to line up with this edge, and to look like a regular foundation wall, especially if it is all covered with stucco or a cement wash. If the footing has a ragged edge because it was poured against earth, the projection may have to be knocked off and the exterior refinished.

Sometimes it is feasible to place beams or channels in grooves cut into an old wall, as shown in Fig. 15-8(*b*). When grouted in and tied together to prevent spreading, they form a strong beam that will prevent local crushing of the wall at a supported point. It is also possible to place continuous beams longitudinally under the needle beams of Sketch (*a*).

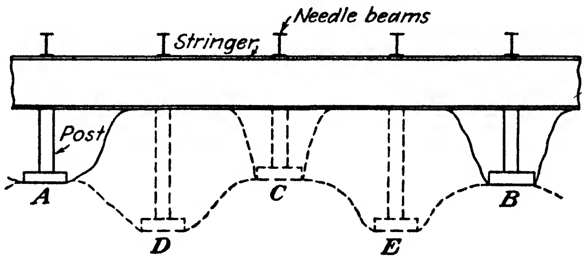


FIG. 15-9. Use of a continuous beam or beams to permit complete excavation for underpinning a wall.

This use of a strong continuous beam is helpful in underpinning, but especially in supporting decking of a street when excavation must be done while traffic continues. The scheme is illustrated in Fig. 15-9. This is to show the case of Fig. 15-8(*a*) where the general area, inside and outside of the building, is to be lowered.

After the stringers of Fig. 15-9 are placed under the needle beams, local pits are dug for supports at *A* and *B*. Mud sills and posts are erected, and the beams are wedged to bearing. The post at *C* is erected and wedged next. The intervening earth is now excavated down to some lower elevation. When the supports at *D* and *E* are in place and wedged, those at *A*, *B*, and *C* are removed, the excavation at these points is carried down still farther. Once more the supports can be installed at these three points so that *D* and *E* can be deepened still farther. These operations can be repeated until the excavation is completed without dropping the structure. After the new foundation is finished, the temporary system can be removed. This method is especially useful when deep excavations must be made and when rock has to be blasted out.

Figure 15-10 shows the use of inclined shoring. The excavation is first made along *ABCD*. The braces *a* are shoved against niches that are cut in the wall. By driving the bottom of a brace inward along timber *b*, the shore itself acts like a wedge. Blocks *c* should be nailed on to prevent slippage. A floor or some lateral support should be opposite or near the top of a to prevent tipping the wall over. Small auxiliary needle beams d are shown also. Their use is to prevent dropping of the bottom portion of the wall. The excavation $CEFG$ and the underpinning can then be completed.

What is called a "figure 4" is illustrated in Fig. 15-11. There are many variations in details. Here is shown the use of a fabricated frame. The

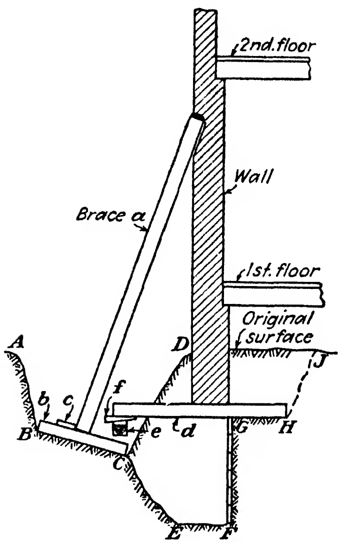


FIG. 15-10. Inclined shoring of a wall.

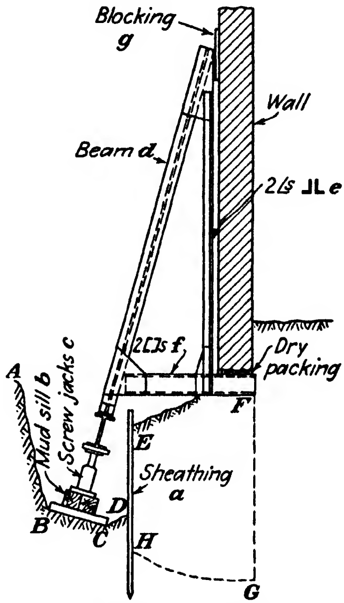


FIG. 15-11. Figure 4 shoring used to hold a wall.

installation of sheathing a and the local excavation $ABCDEF$ constitute the first step. The frame is then erected upon sill b and two or three screw jacks c. The channels f are slipped under the wall or through holes in it. Jacking the frame upward lifts the wall on the cantilevered member f. The excavation $EHGF$ and the rest of the job are then completed. Of course, the wall should be supported against the inward force at the top of the frame.

Instead of a fabricated frame, brace d may be separate and set directly on the mud sill. Member f may be hinged to d in some manner. Long rods with threaded ends, or chains with turnbuckles, may be used instead of a rigid member for e. Tightening the nuts or the turnbuckles then serves as a jack to raise f and the wall.

Sometimes it is not possible to have access to the area outside the building that is to be underpinned. The work is then likely to be even more costly, but some method can usually be found to perform it. The special case in Fig. 15-12 illustrates the use of cantilevers. The excavation $ABCDE$ is made first, with sheathing a in place. Trenches are then gouged out along $JKBFGH$. The supports b and beams c are placed. Posts d are erected at the ends of the beams and braced against the structure so as to hold down

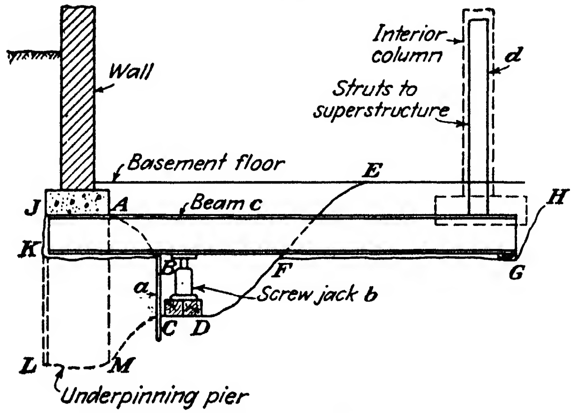


FIG. 15-12. Use of cantilever supports.

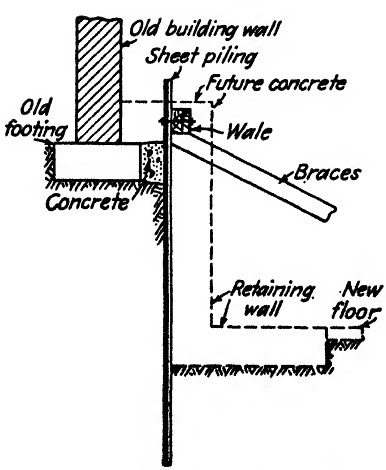


FIG. 15-13. Use of sheet piling left in place.

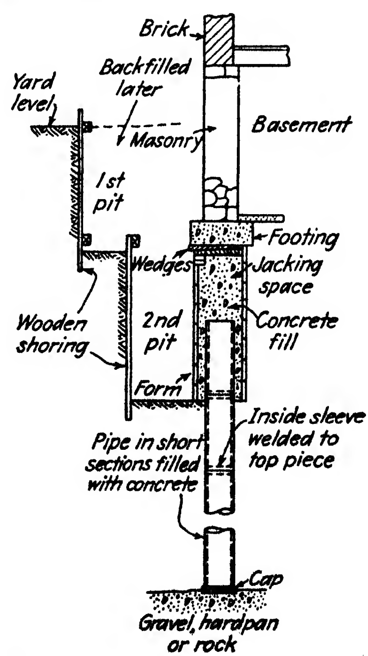


FIG. 15-14. Underpinning with steel-pipe piles jacked in short sections.

the back ends. When the load is transferred to the jacks, the excavation *KLMC* is made and the job is completed similarly to other cases.

An addition was to be made alongside an old wall-bearing building with heavy brick walls and no basement. The addition was to have a basement, as indicated in Fig. 15-13. Steel sheet piling was first driven outside the old wall. A concrete plug was placed between the footing and this piling, as shown. The wale was bolted on and braced horizontally to heavy stakes some distance to the right in the picture. Narrow pits 4 ft. wide

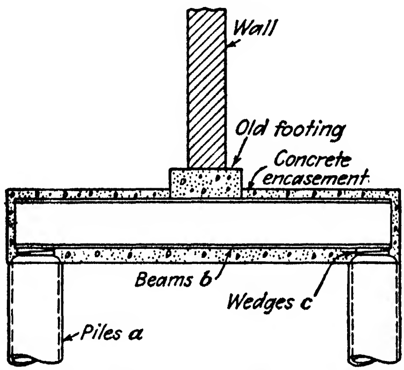
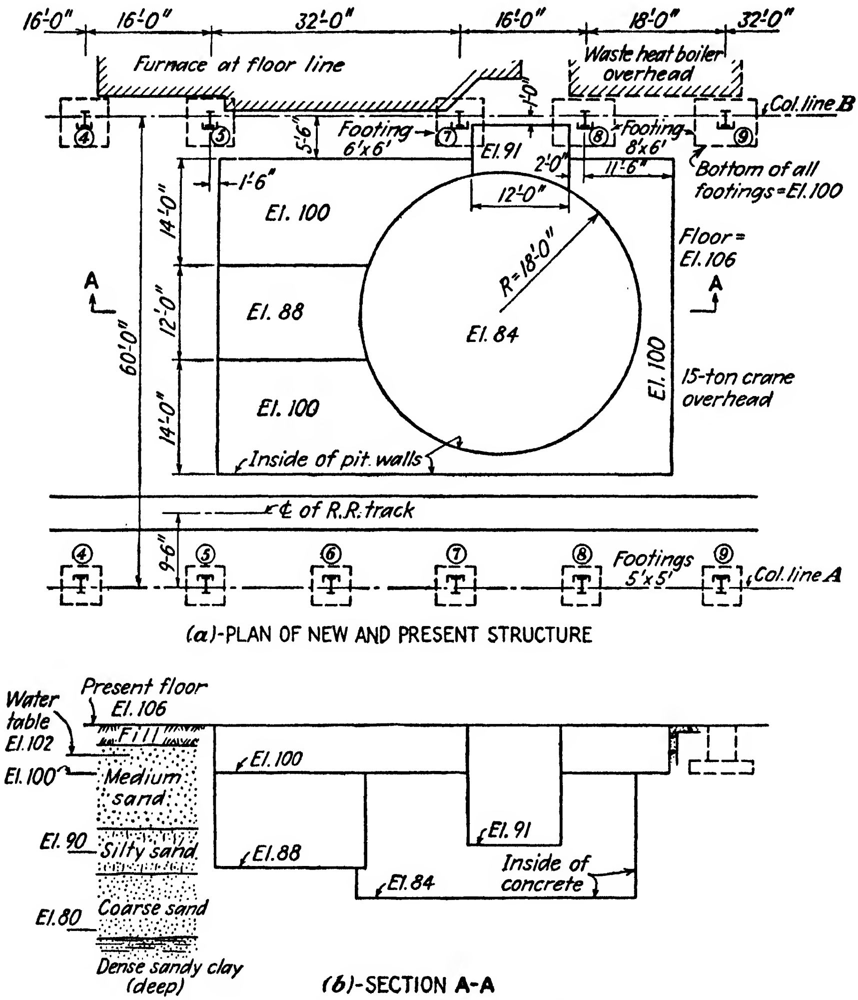


FIG. 15-15. Permanent bridging to adjacent piles.

were then excavated at intervals so that pieces of the retaining wall could be built up to the bottom of the old footing. The concrete was poured directly against the piling. When these blocks were in, the pits were extended so that the wales could be supported by diagonal braces at each piece of wall. These supported the piling while the rest of the excavation and wall were completed. The braces and wales were removed, and the piling was burned off at the top of the old footing. The wall top and a closure piece of concrete then completed the embedment of the piling.

Figure 15-14 shows one way of placing new deep permanent foundations under a building that is settling or is to have greater loads added to it. The local pits are now so deep that they are excavated and sheathed in series as shown. In this case, capped steel pipes were jacked down in short pieces to form piles that were filled with concrete. The jacks were placed on top of a pile section, and they shoved against the wall overhead. A concrete pier was built around and above the pile as an extension thereof. Wedges and packing forced permanent load into each pile. The pipe sections in this case were fitted with sleeves but not welded. Welding may be better where leakage of water is important.

Another method of replacing an old foundation with new piling is indicated in Fig. 15-15. The piles *a* outside may be driven if a pile driver can operate there. Those inside will probably have to be jacked and jetted down. Beams *b* can be inserted one at a time, and then wedged up to bear. When they are all in place, the whole set can be concreted for protection, thus forming a sort of pile cap.



(a)-PLAN OF NEW AND PRESENT STRUCTURE

(b)-SECTION A-A

FIG. 15-16. An underpinning problem at an industrial plant.

Many are the ramifications of underpinning work. Such jobs provide an excellent opportunity to utilize the principles of mechanics, to invent special ways of doing special things, to do them efficiently and economically, and to meet successfully some of the many challenges of the profession.

PROBLEM

Figure 15-16 shows the conditions at an industrial plant located close to a large river. A large concrete pit is to be built to support a new casting wheel that is to receive molten metal directly from the adjacent furnace. The inside outlines of the new construction are shown between column lines *A* and *B* in Sketch (*a*). The elevations of the various levels are shown there as well as in (*b*). The soil conditions are also pictured in (*b*). To be maintained during construction are service of the overhead cranes, the railroad tracks along line *A*, and the tracks outside of line *A* (not shown). The last are close to the columns and constitute a storage yard for cars. How can the construction be conducted safely?

Two 15-ton cranes operate in this aisle of the building. The service of one can be utilized occasionally during the day shifts and continuously for the rest of the time. The furnace and waste-heat boiler will be shut down, but these are very costly and important pieces of equipment and must not be damaged. The furnace is supported by a continuous concrete mat as shown by the hatching. The building is steel frame with corrugated siding and roof and has a concrete floor. There are small grade beams of concrete between the footings of column line *A*.

The concrete for the pit is to be protected on the bottom and sides by two-ply membrane waterproofing.

Suggestions: This problem can be subdivided to make several separate problems. Consider line *A* first. Underpin the necessary footings in this line, assuming their bottoms to be at El. 100. Next, provide the necessary supports for columns B5 and B9. Then plan the underpinning for B7 and B8, remembering that this work must permit the construction of the shallow portion of the pit next to them. Plan the underpinning of the furnace along with that of the adjacent columns, assuming the furnace to be 20 ft. wide. The bottom of the mat under it is at El. 101.5, the tops of the spread footings.

Now plan the construction of the pit itself. Consider both temporary and permanent buoyancy. Assume that the machinery in the circular pit weighs 100 tons, including all accessories and supports.

16

PLANNING THE FOUNDATION FOR A LARGE STACK

16-1. Introduction. Chimneys or stacks are tall heavy structures that may be subjected to large overturning forces caused by wind and earthquakes. It is obvious that they must be supported well. Differential settlement may be very harmful, and the effects are likely to be apparent to all observers.

To illustrate the planning of the foundation for a large stack, the history of a very unusual one will be given. It is one of the most interesting foundation investigations and designs in the author's experience. A detailed description of it will show much that is applicable to ordinary situations as well as much that was extremely special.

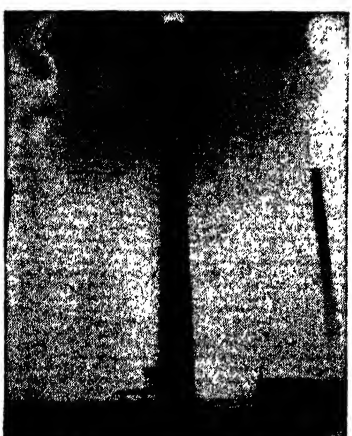


FIG. 16-1. The roaster-reverberatory stack at the Copper Queen Smelter, Douglas, Ariz. It is 540 ft. from the ground line to the top of the stack.

16-2. General conditions. A picture of the completed stack is shown in Fig. 16-1. It is 540 ft. high above ground, a height that is nearly equal to that of the Washington Monument. It is built of reinforced concrete with fiber glass insulation and brick lining. It is located at the Copper Queen Smelter of the Phelps Dodge Corporation at Douglas, Ariz.

During the war the old 300-ft. steel stack at the reverberatory building developed several wrinkles. These were patched with welded stiffeners, but the stack was evidently in serious condition. Because its collapse would cause a complete shutdown of the plant, the officials decided to replace it as quickly as possible.

At the roaster building 620 ft. away, was another old 300-ft. steel stack which had not given evidence of distress but which might do so at any time. Therefore, the corporation decided to build one large stack somewhere between these two buildings and let it handle the gases from the reverberatory furnaces and the roasters through long connecting flues. Thus there would be no interruption of service during the new construction until the connections were made, and this could be done without a shutdown.

A few years previously a reinforced-concrete stack 565 ft. high had been built for the converters. Its height was determined not only for draft but for the delivery of gases high enough in the air to disperse them without annoyance to the surrounding area. This height was set in accordance with the limits specified by the Arizona authorities.

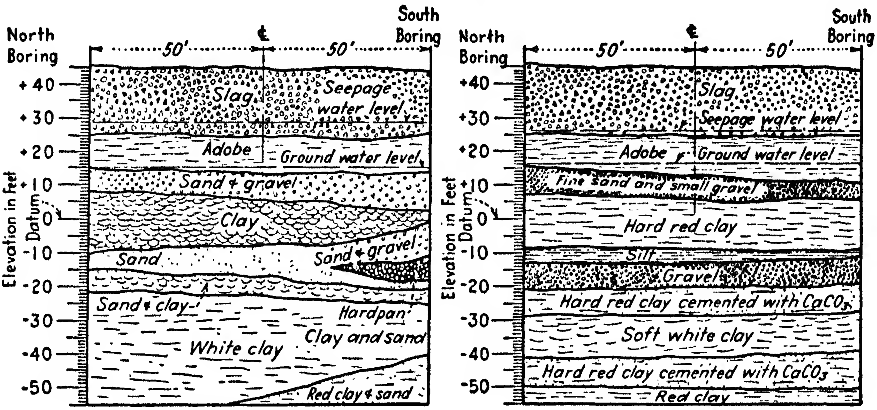
The best location for the new stack seemed to be about midway between the old ones. This was the site of an old slag dump with approximately 20 ft. of slag over the ground. The area was used only as a sort of yard. Being somewhat higher than the land at the converter stack, the height of the new structure was set at 540 ft. in order to have its top level with the other one. Reinforced-concrete construction was also adopted.

Two borings were made at the stack site, using well-drilling equipment, but no undisturbed soil samples were secured. The soil profile shown in Fig. 16-2(*a*) was later plotted from the logs of the borings. However, since the converter stack was only 440 ft. away and was founded upon a strong lens of gravel and caliche, the local authorities did not anticipate serious difficulties. They started the excavation of the slag from the chosen site for the new stack. At the same time, three test pits were dug through 20 ft. of slag and about 10 ft. into the underlying ground. Adobe was found beneath the slag. Considerable water containing sulphuric acid and copper sulphate was encountered near the bottom of the slag. However, the ground water in the sand and gravel below the adobe was pure.

No lens of gravel comparable to that supporting the other large stack was found. Consequently, the first foundation design to be studied was a plain concrete mat 4 to 6 ft. thick placed directly on the adobe. By building the spread footing of the stack on top of this, the former would be raised above the ground-water level where the concrete and reinforcing steel would not be subjected to attack from the acid water. However, a load test on the adobe proved to be unsatisfactory as the material was wet and plastic and flowed out from under the bearing plate. This indicated that the hoped-for bearing resistance of 3 tons per ft.^{2} could not be attained.

On this account, a pile foundation was studied next, using about 570 concrete piles 30 ft. long with an assumed safe load of 30 tons each. In this case the base was to be raised about 10 ft. above the top of the adobe to get it out of danger from the acid-bearing water. The tips of the piles were to penetrate to about El. +5, Fig. 16-2(*a*). When a cross section of this pile design was drawn to show the entire stack, the soil strata, and the relatively "tiny" piles to scale, it became evident that the piles would do little good because they would merely deliver their loads to the tapered clay strata.

A glance at Fig. 16-2(*a*) shows that compression of the varying thicknesses of these clay layers would almost inevitably cause unequal settlements, which would in turn cause tipping of the stack. Inasmuch as the motion of the top would be about seven times as great as the differential settlement



Soil Profile From Original Data (a) **Actual Soil Profile (b)**

FIG. 16-2. Based upon the original borings the soil profile at the left was prepared. Later, after four new borings had been made, the profile at the right was drawn. Comparison of the two profiles shows how misleading inadequate data may be.

of opposite edges of the footing, it was deemed advisable to reexplore the soils before deciding upon the design of the structure. Furthermore, the information shown in Fig. 16-2(*a*) did not seem to be a natural reasonable condition. Even if it proved correct, the characteristics of the soils, particularly the clays, should be ascertained because, if they were highly compressible and if the strata were tapered and deep, it would be almost impossible to prevent serious tilting of the structure.

16-3. Exploration. To gain more complete knowledge of the soil conditions, four new borings were made about 35 ft. from the proposed center for the stack and roughly at the quarter points around the circle. Two others were taken, one about 100 ft. east and another 75 ft. to the southeast of the center of the site. This was done to see if possible neighboring areas might be better.

A north-south profile plotted from the logs of the new borings is shown in Fig. 16-2(*b*). It indicates that the strata are not tapered as shown in Sketch (*a*). The logs of the borings east and southeast of the desired site indicated that nothing was to be gained by shifting the stack in that general direction.

For about 30 ft. below the adobe the various strata proved to be reasonably firm and relatively incompressible; furthermore, the two layers of red clay were quite impervious. The top stratum of sand and gravel consisted largely of fines with some clay and small pebbles. All these strata together formed a sort of earthen slab over 30 ft. thick, which seemed to be fairly strong and reliable in its natural undisturbed state. However, under these strata the 10-ft. layer of soft whitish clay had a consistency and appearance approaching that of well-mixed putty. Its moisture content exceeded 30 per cent, and its bearing value was of low order unless fully restrained.

Although the strata below this clay were strong, they were so deep that piles long enough to reach them would be needlessly expensive. Hence, the idea of using piles was abandoned and study was concentrated on some form of floating foundation that would disturb the natural soil conditions as little as possible.

In the initial study for this foundation, the bottom of the spread footing was located on the first bed of sand and gravel at about El. +13, Fig. 16-2(*b*).

Inasmuch as the 20-ft. layer of slag on the surface of the natural ground had been in place for many years, it seemed probable that appreciable settlement of the underlying strata due to this surcharge had ceased, and that the slag itself would be useful as a blanket in preventing flow of any plastic soils from underneath the structure. Moreover, the laboratory tests of soil samples seemed to show that the materials just under the adobe could safely withstand a maximum pressure of 3.5 tons per ft.^{2} at the edge of the base for a combination of dead load and a wind pressure of 30 p.s.f. of vertical projection of the stack, considering the beneficial effect of the heavy slag overburden.

16-4. Planning of the base. The next thing to be determined was the diameter of the base, using this 3.5 tons per ft.^{2} as the maximum permissible edge pressure. If the adobe and slag were to be backfilled over the spread footing, their weight necessitated a diameter of approximately 110 ft. to meet this condition. As such a large base would be costly, it was decided to eliminate this heavy, useless dead load of the backfill above the footing and support the surrounding soil by a circular concrete wall at the outer edge of the footing, as shown in Fig. 16-3.

The weight of the slag and adobe removed from above the plane of bearing at the bottom of the base would be nearly equal to the dead load of the entire structure and thus produce no material increase in pressure on the underlying strata. The effect of wind was considered to be so temporary, so variable in direction, and so small in magnitude that the resulting inequalities of pressure would not cause the underlying materials to flow or permanently compress, because their impermeability, shearing strength, and cohesive resistance were too great to permit relatively rapid changes, The problem of protection against the acid-bearing ground water was solved by using membrane waterproofing, as illustrated in Fig. 16-3. If the chemicals attacked the concrete outside the waterproofing it seemed that nothing worse could result than its conversion to a sort of sand and gravel layer still capable of transmitting the necessary pressures.

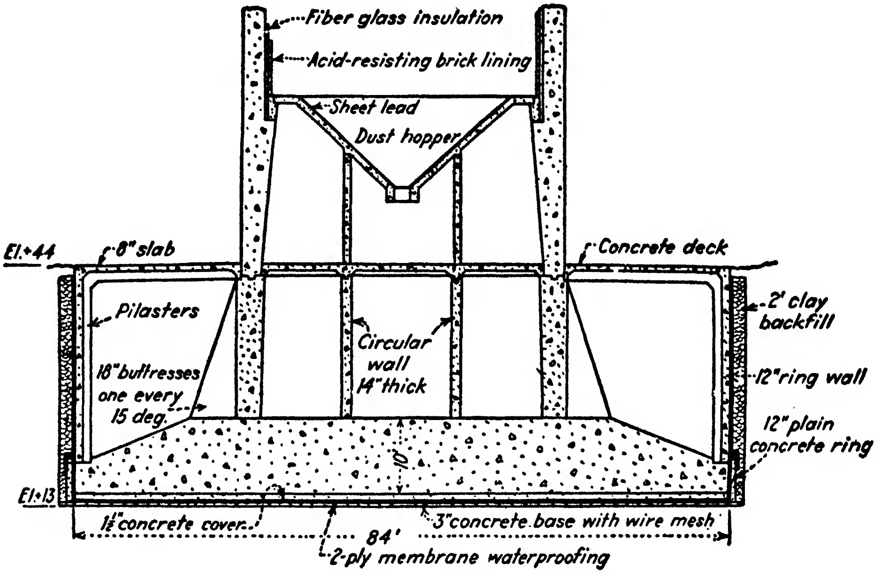


FIG. 16-3. The original floating foundation designed for the stack called for an 84-ft.-diameter spread footing located on sand and fine gravel 30 ft. below the surface.

16-5. Difficulties encountered. The contract for the foundation was let on a unit-price basis and was handled as a separate job. The remainder of the work, the stack above El. +44, was let as a lump-sum contract to a chimney specialist. Work of completing the excavation was started. When the slag was practically all removed and some of the adobe had been excavated to about El. +17, ground water began to soften the adobe so badly that the digging of two sumps just outside of the future rim of the base was started to permit pumping out the water as fast as it entered. By the time these sumps had been excavated to El. +14, the ground-water pressure below the adobe caused the bottoms to blow up and the water to rise to about El. +15, which was 2 ft. above the proposed elevation of the bottom of the base. Fear of complete disintegration of the bottom of the pit, if carried to the intended elevation, caused cessation of the work.

The ground water in the slag was so heavily charged with acid that lime had to be dumped in the sumps to neutralize it and thus avoid damage to the pumps. Even then both pumps broke down on the same day. The water soon accumulated, as shown in Fig. 16-4.

16-6. Studies of remedies. Engineers representing the foundation and chimney contractors combined their efforts with those of the engineers at the plant and those engaged by the owners. As in such emergencies, all joined in the effort to devise the best solution to their problem.

The first problem was to intercept the ground water at the bottom of the slag and carry it to the sumps directly without letting it reach the excavated area. Although the slag was broken into small pieces, the particles

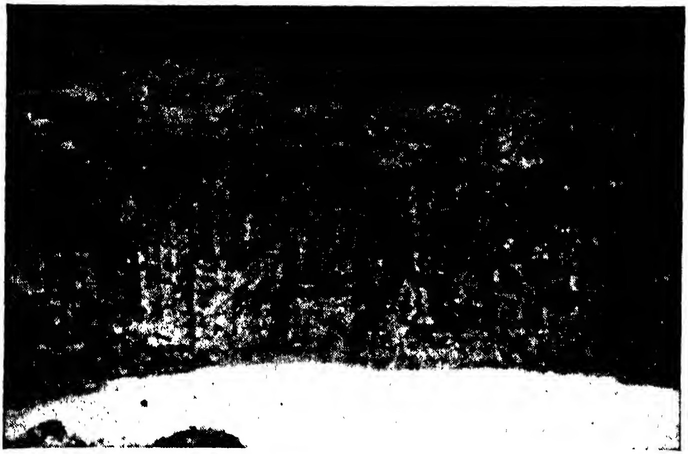


FIG. 16-4. A discouraging sight. Looking down at the accumulating water in the excavation after the pumps broke down. Slag is shown on top of the adobe.

seemed to be well interlocked so as to give the slag layer considerable shearing strength. Therefore, it was considered safe to undermine the edge and permit installation of corrugated-iron troughs to intercept the flow and carry it to the sumps. These were successfully completed and are pictured in Fig. 16-5. As soon as the pumps were again working, the process of drying up the excavation was started.

In the meantime, the elevation of the bottom of the base was tentatively ordered raised 3 ft. to El. +16, that being slightly above the level reached by the ground water rising in the sumps, and just under the deepest part of the excavation already made. It was thought that the weight and impervious nature of the adobe would be sufficient to prevent blowing up of the bottom when the excavation to that depth was completed.

The use of wellpoints was considered to remove the ground water from the top stratum of sand and gravel shown in Fig. 16-2(*b*) so that the base could be kept at its original elevation, but the plan was abandoned for the following reasons: That layer contained a large amount of very fine material which might be washed out by the flow of water to the wellpoints, the quantity of water to be handled was completely unknown, the laboratory tests indicated that the action of the artesian water was likely to disintegrate

(a) Cutting continuous shelf under edge of slag. Entering water is visible. The white patches in the bottom are lime used to neutralize the acid.

(b) Launder in place at northeast side. A sump is in the left foreground. The excavated area is beginning to dry up.

FIG. 16-5. Method of intercepting acid-bearing water at the bottom of slag.

this layer seriously, the performance of the soil at the bottoms of the sumps seeming to substantiate this latter conclusion. Furthermore, it seemed wise to avoid doing anything that might disturb the lower strata.

16-7. Testing the soil. The plan to raise the elevation of the bottom of the base meant that at least a part of the base would have to rest on a few feet of adobe of questionable bearing capacity. Hence, load tests of this material were started.

Inasmuch as the design of the foundation was based upon utilizing the holding-down pressure of the materials outside of the footing itself, the tests were to take this feature into account. If they were made directly on the surface of the adobe, it seemed that such a plastic material would certainly flow out from under the loaded areas and that any such test would show

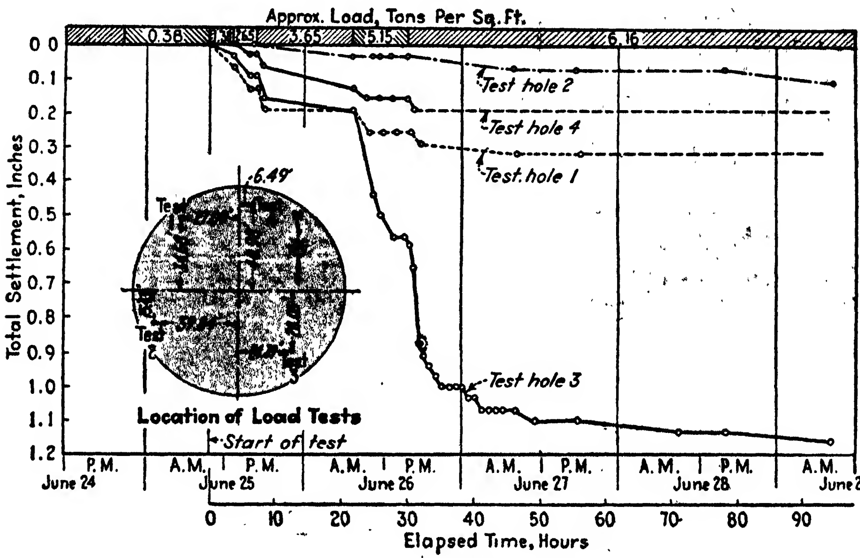


FIG. 16-6. Raising the bottom of the base 3 ft. required that the structure rest on adobe of uncertain bearing capacity, which made load tests of the adobe desirable. Four tests, spotted as shown at the left, supplied the data above.

far less trustworthy results than would a test made with some reasonable depth of material surrounding the loaded area.

Therefore, the following procedure was ordered: (1) Make the tests so that the loaded area would be at El. +16, the assumed new elevation of the bottom of the concrete; (2) test a loaded area of 2 ft.^{2}; (3) apply the load at the bottom of a hole 3 ft. deep below the adobe surrounding that vicinity; (4) make four individual tests, locating them as near the quarter points of the pit's perimeter as possible under existing conditions; (5) take load-settlement readings for all tests; (6) assume that settlement of 1 in. constituted failure of the soil; and (7) allow sufficient time to assure that the settlements under each increment of load practically ceased before applying the next one.

The points for testing were chosen at the locations shown in Fig. 16-6. The immediate vicinity of each was leveled off at El. +19.

The plant engineers made the loading masts, which consisted of a circular steel plate, 2 ft.² in area, at the bottom of a 12-in. steel pipe surmounted by a timber loading platform on a steel frame. The holes in which the tests were made were lined by sinking 20-in. steel pipes to the desired depth, after which the bottoms were smoothed off and a little dry sand sprinkled

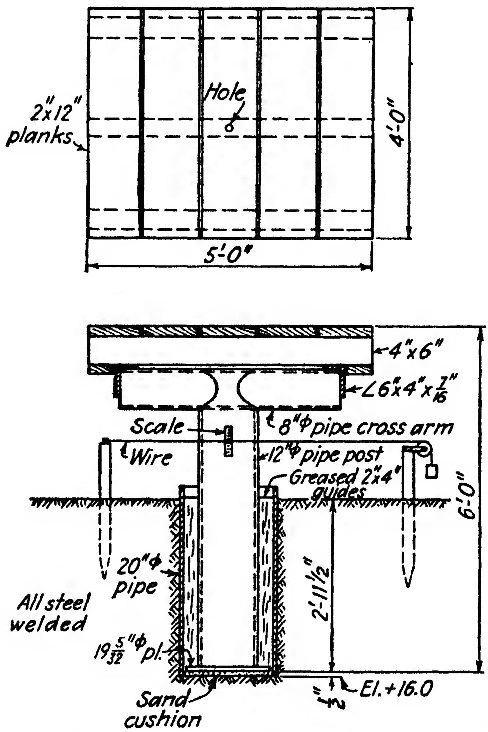


FIG. 16-7. Platform and general arrangement for a loading test of soil.

in and leveled. Loading masts were then erected, plumbed, and rotated to get even bearing on the sand. Each mast was held vertically by installing six greased 2- by 4-in. wedges cut to proper size. These bore against the outer shell but did not hamper vertical motion of the mast. Some of the construction is pictured in Fig. 16-7.

In order to get settlement readings, a scale was attached to each mast, then a fine wire was stretched across two stakes and counterweighted. To check the scale readings and to avoid losing the value of a test in case something happened to a scale or wire, a hole was cut in the wooden platform so that a level rod could be set on the top of a cross arm over the center of the mast.

The material found in the excavation of the test holes was about as follows:

No. 1: Adobe changing to a mixture of small wet gravel and dark coarse sand near the bottom of the hole.

No. 2: Reddish soil, relatively dry and seeming to contain a sort of skeleton of cementing materials. It had hard spots and veins that appeared more whitish than the rest of the ground.

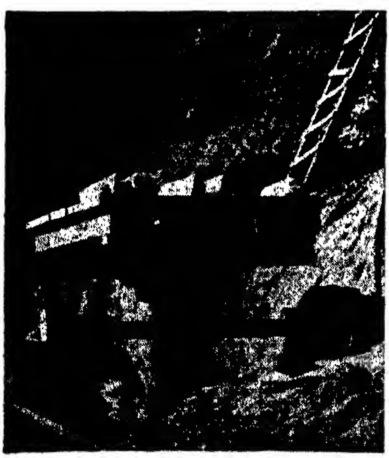
No. 3: Dense dark-red plastic adobe for the full depth. The material was wet but relatively impermeable. This soil was far softer than that encountered in the other holes.

No. 4: The bottom consisted of a mixture of red clay and coarse sand, but it was not so coarse and wet as the soil in No. 1.

These findings seemed to indicate that the bottom of the adobe stratum was sloping and that the base of the stack would rest upon plastic material on one side and firmer soils on the other. This was even more disturbing than would have been a thin but uniform layer of adobe under the structure because of the likelihood of uneven settlement.

When the test equipment was all set in place, an initial load of about 500 p.s.f. was applied, using pig lead as shown in Fig. 16-8. This load was allowed to remain all night to be sure that all slack was taken up. Thereafter, readings were taken just before and immediately after the application of each major increment of load.

Time-settlement curves for all the tests are shown in Fig. 16-6. Test pit No. 3 proved to be the critical one. The soil carried the load fairly well until it was increased to 5 tons per ft.^{2}, whereupon the adobe started to yield badly. Its settlement was recorded every 2 or 3 hrs. all through the following night. The other three test areas were loaded later to 6 tons per



(a) Initial load in place to take up slack.

(b) Platform fully loaded for 6 tons per sq. ft. on bearing area.

FIG. 16-8. Pictures of loading at test No. 2.

ft.[2] Although they held fairly well, it was evident that test pit No. 3, which represented the general condition along the east and south sides, would control the maximum allowable pressure to be assumed in revising the design, its ultimate load being taken as 5 tons per ft.[2] The test loads were left on for another 48 hr., to make sure that serious progressive settlements did not occur. Furthermore, four additional test pits were dug to locate the full extent of the adobe at the elevation of the bottom of the base. In general, the results of the field tests of the adobe were higher than those made in the laboratory.

16-8. Final design. The entire problem was now reviewed. The urgency of the work, because of the dangerous condition of the reverberatory stack, and the apparent economy of a single stack made it seem advisable to modify the design to make the structure safe at the chosen location. Of course, the existence of relatively firm materials in one-half of the excavation, whereas the other half was weak and very plastic, seemed to be just about as bad a condition as one could find for the foundation of such a heavy slender structure. Obviously, the diameter of the base should be increased.

The maximum allowable pressure for dead loads was assumed to be 2 tons per ft.[2] This was to be practically the same as the estimated weight of the materials that would be removed above El. +16, the selected position for the bottom of the base. The maximum allowable total pressure under one edge of the base for dead load plus wind was set at $2\frac{3}{4}$ tons per ft.[2], giving a safety factor of nearly two with respect to the failure of the soil under test.

Considering the fact that the diameter of the excavation existing in the slag was about 100 ft., a total diameter of the base of 99 ft. was chosen, tentatively found to be satisfactory, and approved as the starting point for the new design. The final result is shown in Fig. 16-9. Since the weight of the base would be about half of the total weight of the stack, it seemed probable that this load would take up a lot of the settlement due to recompression of the soil if it tended to rebound somewhat after the excavation was made and then to settle again as the load was applied.

There was a real possibility that the almost vertical face of the adobe would not continue to stand up under the slag if softened by ground water or rain. Therefore, the concrete protection below the membrane waterproofing and the ringwall outside of the edge of the footing were made heavy enough to form a strong "pan" at the bottom of the excavation. Once they were in, the bottom would be protected, and the ring would help hold the adobe in place.

A detail of this very important feature is shown in Fig. 16-9. The ring was reinforced to act as a retaining wall; the edge of the slab was also reinforced so that it cantilevered beyond the edge of the heavy footing, thereby making the full 99-ft. diameter effective. The work was carried on in individual sectors in order to minimize the area exposed at any one time. After completion of the pan, the greatest danger was over.

A few of the features shown in Fig. 16-9 should be noted. To reduce the cantilevered portion of the footing, the lower part of the stack was belled out in a solid ring instead of using buttresses as in the earlier designs, the

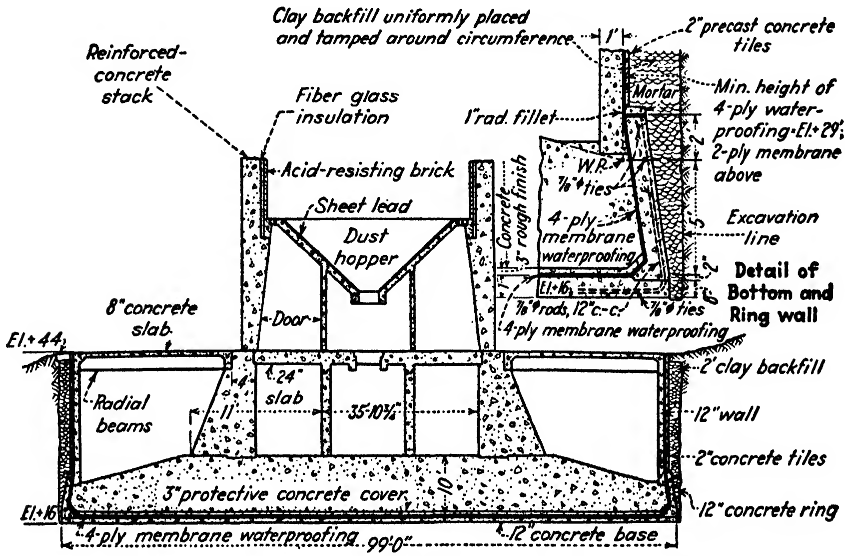


FIG. 16-9. Details of the foundation as finally constructed. The base was designed to support the stack without increasing the load on the supporting soil. Special care was taken to minimize possible damage to the concrete from the acid-bearing ground water.

top of the belled portion being supported by a 24-in. concrete slab to resist the inward thrusts at the top. The "tank" wall was supported laterally by the concrete deck.

The membrane waterproofing of the pan was applied on the inside whereas that above it was placed after the completion of the concrete ringwall. The upper waterproofing was protected on the outside by large precast concrete tiles, unreinforced and 2 in. thick, laid in mortar. The space between them and up to the face of the slag was backfilled with clay to minimize the penetration of water. A sump was built in the top of the footing in case leaks occurred. A manhole and ladder were provided to permit inspection.

After completion of the stack, level readings were taken at four points on the perimeter of the base at the top of the ringwall. These were repeated at intervals. At the end of 2 years the total settlement was of the order of ¼ in., and it was substantially uniform.

16-9. Details of the footing. Regardless of the special details of the base as a whole, the design of the footing of this stack was rather typical except for the size of the footing. Naturally, for such a structure, the design of the footing becomes a very serious matter. The assumptions and procedure used will be explained.

The dead load of the completed structure was estimated carefully from the detail drawings. Then the overturning moment about the center of the bearing area (bottom of the pan) was computed upon the assumption that a wind pressure of 30 p.s.f. acted upon the entire projected area of the stack above the surface of the ground. The cylindrical surface would have some effect in streamlining the structure, but it was not relied upon as a means of reducing the overturning moment except that this feature made it seem unnecessary to apply even larger pressures near the top to allow for unusual gales. The passive resistance of the earth against the side of the footing and the "tank" walls was disregarded.

Arbitrarily, the footing was made very thick near the rim of the belled base—thick enough to ensure safe shearing stresses without web reinforcement and to attain great stiffness. The top of the footing was sloped as much as seemed desirable without handicapping the placement of the concrete, and to keep an edge thick enough to support the adobe under the slag.

The footing beyond the belled base was analyzed as a series of sectors having a width of 1 ft. at the perimeter of the bell circle, and having radial sides as shown in Fig. 16-10(*b*). The pressure diagram was found for dead load and wind. A typical sector on the high-pressure side was analyzed as a beam cantilevered from the bell and having a uniformly varying pressure under it acting upon the flaring area pictured in Fig. 16-10(*b*). The weight of the footing and the reaction of the tank wall were considered as relieving upward bending. The magnitude of the upward pressure minus the footing weight was computed, and the point of application of its resultant determined. The shear and bending moment at the face of the bell were then estimated and applied to the assumed 1-ft. section. The reinforcement was determined accordingly, and the bond and shear were checked as for a deep cantilevered beam. All sections were then made to have the same strength.

Greatly abbreviated computations for the pressure under the base and for the reinforcement are given, using a 99-ft. circular bearing area:

Part	*V, tons*	*H, tons*		*M, ft.-tons*
Completed shaft to El. +44...	7,112	284.8		64,887
Base to El. +16.............	7,751		$(284.8 \times 28) =$	7,971
	14,863			72,858

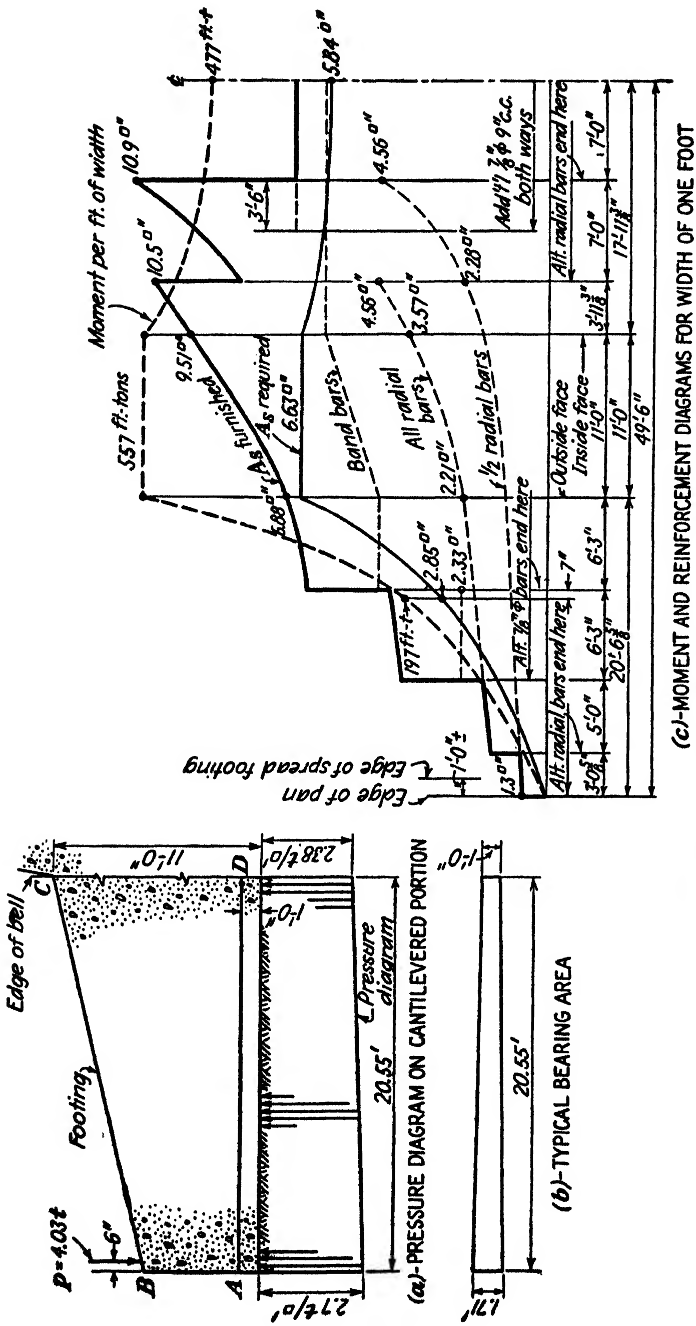


FIG. 16-10. Dimensions, pressures, moments, and reinforcement for footing of the stack.

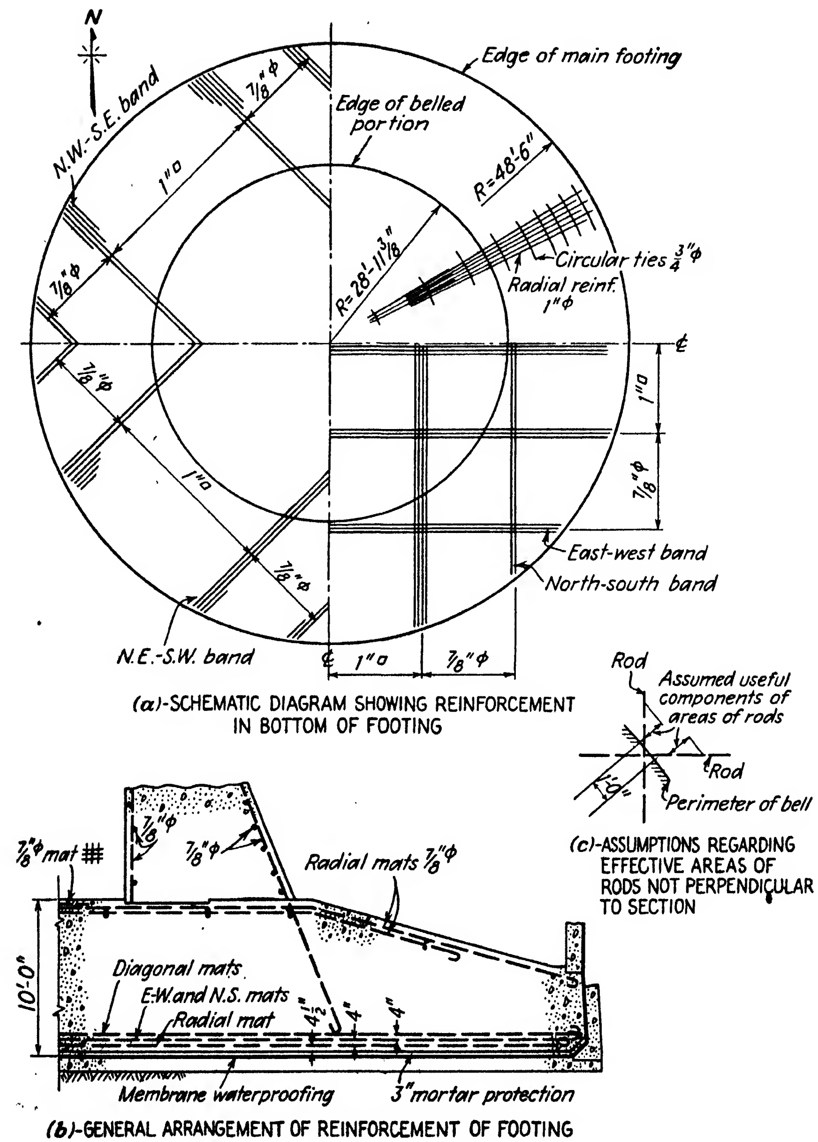
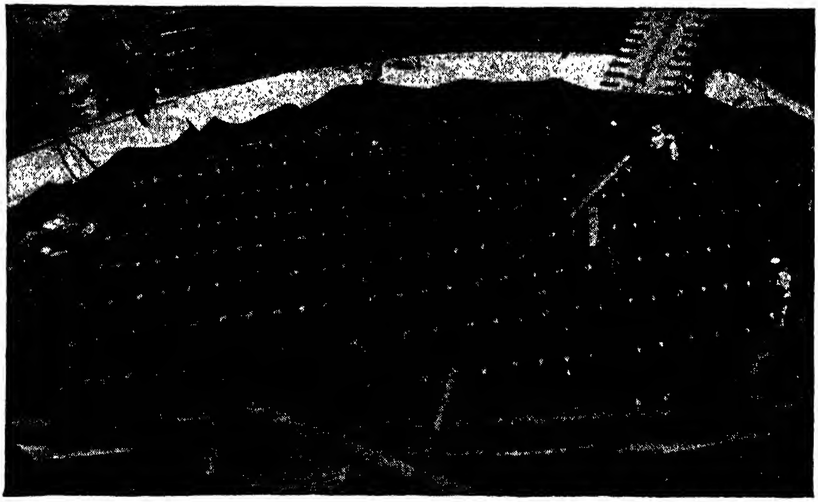
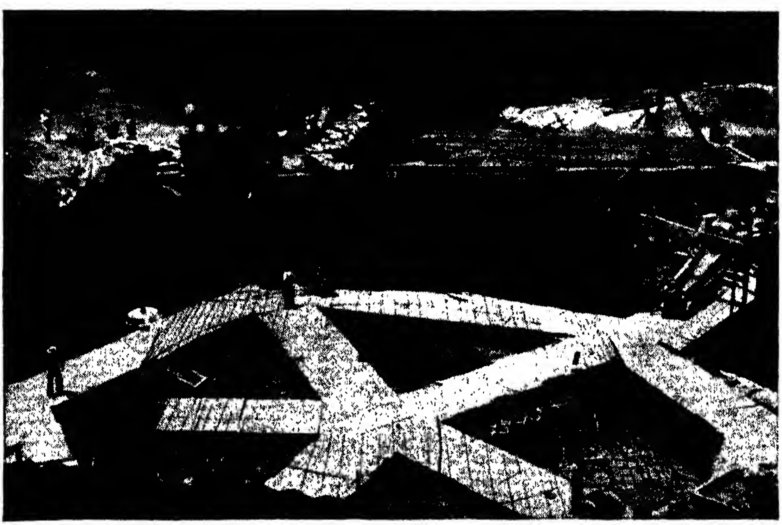


FIG. 16-11. Details of the footing and its reinforcement.



(a) Placing the bottom reinforcing. The waterproofing fabric is shown bent down temporarily over the rim of the "pan."



(b) Reinforcement in place, with walkways for concrete buggies. Rods for the bell are omitted temporarily at two walkways.

FIG.16-12. Construction of the stack footing.

Soil pressure:

$$\text{Dead load} = \frac{14{,}863}{7{,}697} = 1.93 \text{ tons per ft.}^2$$

$$\text{Dead load and wind} = 1.93 \pm \frac{72{,}858}{0.785 \times 49.5^3} = \begin{matrix} 2.70 \text{ tons per ft.}^2 \\ 1.16 \text{ tons per ft.}^2 \end{matrix}$$

The critical pressure condition is shown in Fig. 16-10(a). Net shear at $CD = 50.1$ tons; net moment at $CD = 557$ ft.-tons.

$$v_L \text{ at } CD = \frac{50.1 \times 2{,}000}{12 \times 0.9 \times 112} = 83 \text{ p.s.i. (approx)}$$

$$A_s \text{ at } CD = \frac{557 \times 2{,}000 \times 12}{20{,}000 \times 0.9 \times 112} = 6.63 \text{ in.}^2$$

The moment diagram and the steel areas provided per foot of width at various sections are shown in Fig. 16-10(c).

The arrangement of the reinforcement was entirely a practical matter. It is obvious that all the bars could not be set in a radial position. Therefore, four broad bands of rods were placed at 45°, as shown in Fig. 16-11(a). Another layer was composed of radial rods that were terminated near the center when they converged to a minimum spacing for placing the concrete. The effective area of steel to resist bending at any foot of the perimeter of the bell was considered to be the full cross section of the radial bars (averaged per foot) and the components in a radial direction of the areas of those bars that were not in a radial position at the section being considered. A typical case is illustrated in Fig. 16-11(c). Ties were used around the edges of the footing as indicated in the figure. For smaller footings, two bands at 90° and the set of radial rods would probably be sufficient for the bottom. Also shown is the general arrangement of the reinforcement in the top of the footing and in the bottom of the bell. This is used to provide resistance to the effects of tension at the windward edge of the bottom of the shaft.

APPENDIX

Table 1. Approximate Formulas for Analysis of Reinforced-concrete Members

$M = A_s f_s jd \qquad f_s = \dfrac{M}{A_s jd} \qquad A_s = \dfrac{M}{f_s jd}$	For rectangular and T beams
$M = \frac{1}{2} f_c kjbd^2 \qquad f_c = \dfrac{2M}{kjbd^2}$	For rectangular beams
$k = \sqrt{2pn + (pn)^2} - pn \qquad j = 1 - \dfrac{k}{3}$	For rectangular beams
$u = \dfrac{V}{(\Sigma o)jd} \qquad v_L = \dfrac{V}{bjd} \qquad v_T = \dfrac{V}{bkd}$	For rectangular beams and ends of continuous beams

Approximate values of coefficients: $k = 0.3$, $j = 0.9$.

Symbols:

- A_s = area of steel in tension, in.^{2}
- b = width of rectangular beam or stem of T beam, in.
- d = depth from compression face of beam or slab to the center of gravity of the longitudinal tensile reinforcement, in.
- f_c = compressive unit stress in extreme fiber of concrete, p.s.i.
- f_s = tensile unit stress in longitudinal reinforcement, p.s.i.
- j = ratio of distance between centroid of compression and *c.g.* of tensile steel to depth d.
- k = ratio of distance between compression face of beam or slab and neutral axis to depth d
- M = internal resisting moment or applied bending moment, in.-lb.
- n = ratio of modulus of elasticity of steel to that of concrete
- p = ratio of area of tensile reinforcement to that of effective area of concrete bd (for beams and slabs)
- u = average bond unit stress on tensile reinforcement, p.s.i.
- v_L = computed shearing unit stress based upon depth jd, and assumed to be a measure of the corresponding diagonal tension, p.s.i.
- v_T = computed shearing unit stress based upon depth kd, and assumed to be a measure of punching shear, p.s.i.
- V = total transverse shear in the member at cross section being considered, lb.
- Σo = total surface area of the tensile reinforcement per inch of length at cross section being considered, in.^{2}
- kip = 1,000 lb. (often abbreviated as k)

Table 2. Allowable Values of Unit Stresses Assumed for Analysis and Design of Substructures, Assuming Ultimate Compressive Stress = 3,000 p.s.i.*

$f_c = 1{,}200$ p.s.i.	$v_L = 90$ p.s.i.
$f_s = 18{,}000$ p.s.i.	$v_T = 600$ p.s.i.
$u = 150$ p.s.i.	$n = 10$

* See Table 1 for meaning of the symbols.

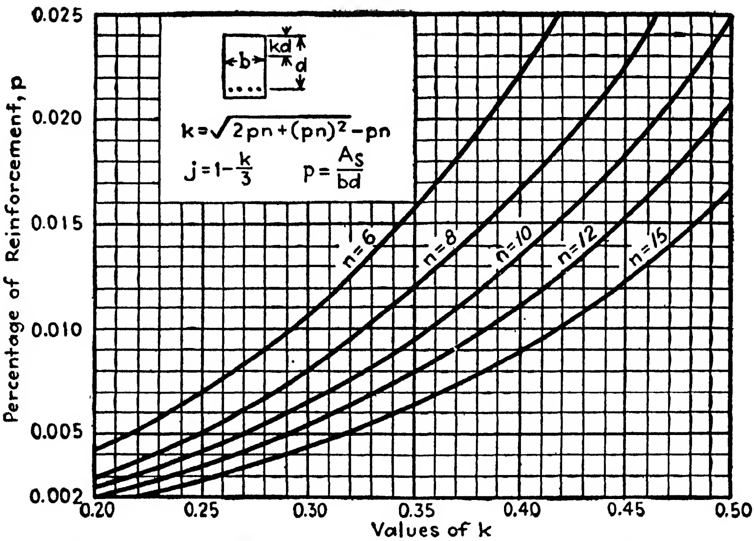


FIG. 1. Location of neutral axis of rectangular beams with tensile steel only.

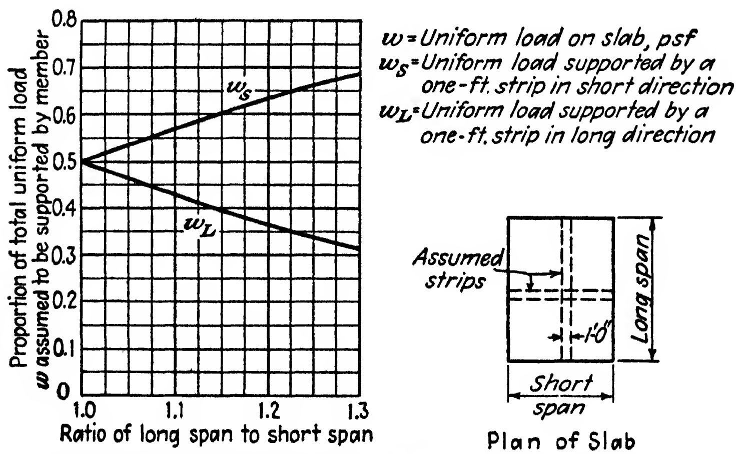
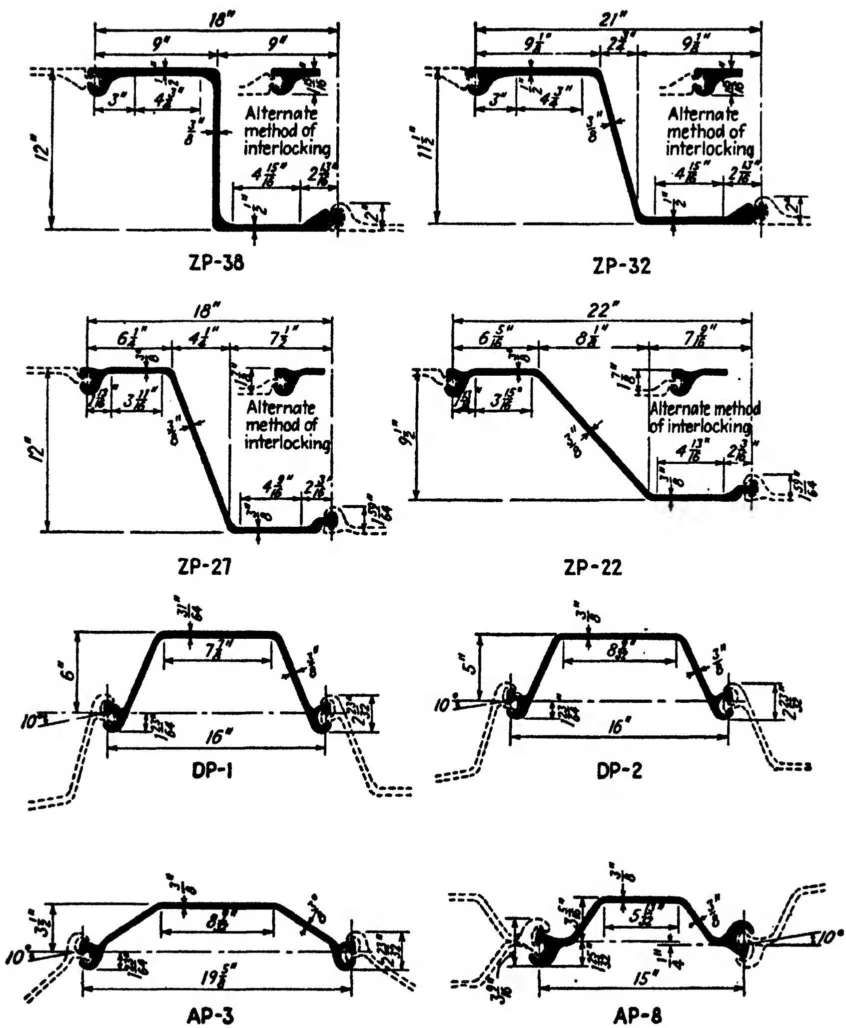


FIG. 2. Assumed distribution of uniform load on a rectangular panel supported on four sides and reinforced in long and short directions.

TABLE 3. AREAS AND PERIMETERS OF RODS PER FOOT OF SLAB

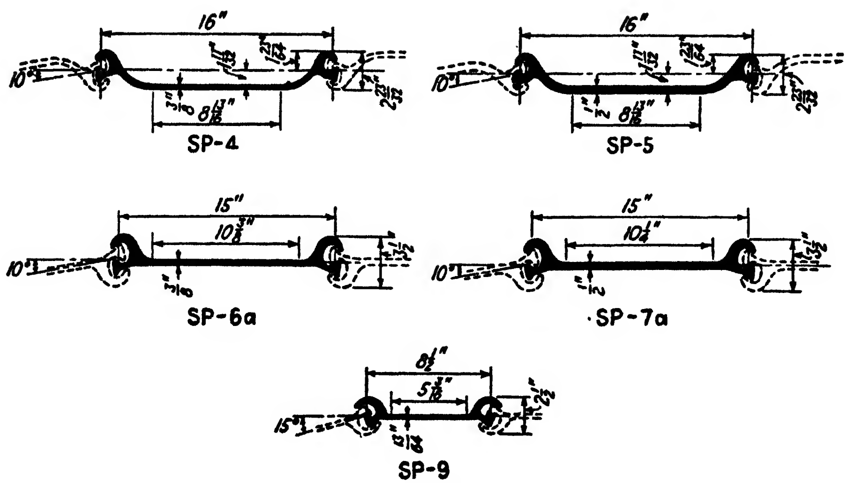
Spacing, in.	Size of rod																					
	¼″ φ		⅜″ φ		½″ φ		½″ □		⅝″ φ		¾″ φ		⅞″ φ		1″ φ		1″ □		1⅛″ □		1¼″ □	
	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo
2	**0.30**	*4.68*	**0.66**	*7.08*	**1.20**	*9.42*	**1.50**	*12.00*	**1.86**	*11.76*												
2½	**0.24**	*3.74*	**0.53**	*5.66*	**0.96**	*7.54*	**1.20**	*9.60*	**1.49**	*9.41*	**2.11**	*11.33*										
3	**0.20**	*3.12*	**0.44**	*4.72*	**0.80**	*6.28*	**1.00**	*8.00*	**1.24**	*7.84*	**1.76**	*9.44*	**2.40**	*11.00*	**3.16**	*12.56*	**4.00**	*16.00*				
3½	**0.17**	*2.67*	**0.38**	*4.05*	**0.69**	*5.39*	**0.86**	*6.86*	**1.06**	*6.72*	**1.51**	*8.09*	**2.06**	*9.43*	**2.71**	*10.77*	**3.43**	*13.72*	**4.35**	*15.43*		
4	**0.15**	*2.34*	**0.33**	*3.54*	**0.60**	*4.71*	**0.75**	*6.00*	**0.93**	*5.88*	**1.32**	*7.08*	**1.80**	*8.25*	**2.37**	*9.42*	**3.00**	*12.00*	**3.81**	*13.50*	**4.68**	*15.00*
4½	**0.13**	*2.08*	**0.29**	*3.15*	**0.53**	*4.19*	**0.67**	*5.33*	**0.83**	*5.23*	**1.17**	*6.29*	**1.60**	*7.33*	**2.11**	*8.37*	**2.67**	*10.67*	**3.39**	*12.00*	**4.16**	*13.33*
5	**0.12**	*1.87*	**0.26**	*2.83*	**0.48**	*3.77*	**0.60**	*4.80*	**0.74**	*4.70*	**1.06**	*5.66*	**1.44**	*6.60*	**1.90**	*7.54*	**2.40**	*9.60*	**3.05**	*10.80*	**3.74**	*12.00*
5½	**0.11**	*1.70*	**0.24**	*2.57*	**0.44**	*3.43*	**0.55**	*4.36*	**0.68**	*4.28*	**0.96**	*5.15*	**1.31**	*6.00*	**1.72**	*6.85*	**2.18**	*8.73*	**2.77**	*9.82*	**3.40**	*10.91*
6	**0.10**	*1.56*	**0.22**	*2.36*	**0.40**	*3.14*	**0.50**	*4.00*	**0.62**	*3.92*	**0.88**	*4.72*	**1.20**	*5.50*	**1.58**	*6.28*	**2.00**	*8.00*	**2.54**	*9.00*	**3.12**	*10.00*
6½	**0.09**	*1.44*	**0.20**	*2.18*	**0.37**	*2.90*	**0.46**	*3.69*	**0.57**	*3.62*	**0.81**	*4.36*	**1.11**	*5.08*	**1.46**	*5.80*	**1.85**	*7.38*	**2.34**	*8.31*	**2.88**	*9.23*
7	**0.09**	*1.34*	**0.19**	*2.02*	**0.34**	*2.69*	**0.43**	*3.43*	**0.53**	*3.36*	**0.75**	*4.05*	**1.03**	*4.71*	**1.35**	*5.38*	**1.71**	*6.86*	**2.18**	*7.71*	**2.67**	*8.57*
7½	**0.08**	*1.25*	**0.18**	*1.89*	**0.32**	*2.51*	**0.40**	*3.20*	**0.50**	*3.14*	**0.70**	*3.78*	**0.96**	*4.40*	**1.26**	*5.02*	**1.60**	*6.40*	**2.03**	*7.20*	**2.50**	*8.00*
8	**0.08**	*1.17*	**0.16**	*1.77*	**0.30**	*2.36*	**0.38**	*3.00*	**0.46**	*2.94*	**0.66**	*3.54*	**0.90**	*4.12*	**1.18**	*4.71*	**1.50**	*6.00*	**1.90**	*6.75*	**2.34**	*7.50*
9	**0.07**	*1.04*	**0.15**	*1.57*	**0.27**	*2.09*	**0.33**	*2.67*	**0.41**	*2.61*	**0.59**	*3.15*	**0.80**	*3.67*	**1.05**	*4.19*	**1.33**	*5.33*	**1.69**	*6.00*	**2.08**	*6.66*
10	**0.06**	*0.94*	**0.13**	*1.42*	**0.24**	*1.88*	**0.30**	*2.40*	**0.37**	*2.35*	**0.53**	*2.83*	**0.72**	*3.30*	**0.95**	*3.77*	**1.20**	*4.80*	**1.52**	*5.40*	**1.87**	*6.00*
11	**0.05**	*0.85*	**0.12**	*1.29*	**0.22**	*1.71*	**0.27**	*2.18*	**0.34**	*2.14*	**0.48**	*2.57*	**0.65**	*3.00*	**0.86**	*3.43*	**1.09**	*4.36*	**1.39**	*4.91*	**1.70**	*5.46*
12	**0.05**	*0.78*	**0.11**	*1.18*	**0.20**	*1.57*	**0.25**	*2.00*	**0.31**	*1.96*	**0.44**	*2.36*	**0.60**	*2.75*	**0.79**	*3.14*	**1.00**	*4.00*	**1.27**	*4.50*	**1.56**	*5.00*

TYPES, DIMENSIONS, AND PROPERTIES OF SECTIONS



Sections ZP-38 and ZP-32 will interlock with each other.
Sections DP-1, DP-2, AP-3, SP-4 and SP-5 will interlock with each other.
Sections SP-6a, SP-7a and AP-8 will interlock with each other.
Section SP-9 will not interlock with any other Bethlehem section.

FIG. 3. Data regarding steel sheet piling. (*Courtesy of Bethlehem Steel Company.*)



Section No.	Area, sq. in.	Width, in.	Weight, lb.		Moment of inertia, in.^{4}	Section modulus, in.^{3}	
			Per lin. ft. of bar	Per sq. ft. of wall		Single section	Per lin. ft. of wall
ZP-38	16.77	18	57.0	38.0	421.2	70.2	46.8
ZP-32	16.47	21	56.0	32.0	385.7	67.0	38.3
ZP-27	11.91	18	40.5	27.0	276.3	45.3	30.2
ZP-22	11.86	22	40.3	22.0	167.4	34.8	19.0
DP-1	12.56	16	42.7	32.0	87.0	20.4	15.3
DP-2	10.59	16	36.0	27.0	53.0	14.3	10.7
AP-3	10.59	19⅝	36.0	22.0	26.0	8.8	5.4
SP-4	8.99	16	30.7	23.0	5.5	3.2	2.4
SP-5	10.98	16	37.3	28.0	6.0	3.3	2.5
SP-6a	10.29	15	35.0	28.0	4.6	3.0	2.4
SP-7a	11.76	15	40.0	32.0	4.6	3.0	2.4
AP-8	11.41	15	38.8	31.0	24.2	8.9	7.1
SP-9	4.38	8½	14.9	21.0	1.0	1.0	1.4

When minimum ultimate interlock strength is specified for SP-4, SP-5, SP-6a and SP-7a, they can be furnished with the following strengths at the published extra charge: SP-4 and SP-5 = 12,000 lb. per in., SP-6a and SP-7a = 16,000 lb. per in.

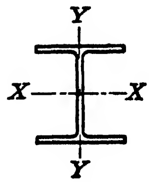
Values listed apply only to the interlocking joints and not to the entire cross section of the pile. Sections SP-4 and SP-5, when subjected to direct tension as in cellular construction, should be investigated for web extension which will occur before the interlock strength is developed.

FIG. 3. (*Concluded.*)

TABLE 4. AREAS AND PERIMETERS OF RODS

No. of rods	Size of rod																					
	$\frac{1}{4}''\phi$		$\frac{3}{8}''\phi$		$\frac{1}{2}''\phi$		$\frac{1}{2}''\square$		$\frac{5}{8}''\phi$		$\frac{3}{4}''\phi$		$\frac{7}{8}''\phi$		$1''\phi$		$1''\square$		$1\frac{1}{8}''\square$		$1\frac{1}{4}''\square$	
	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo	A_s	Σo
1	0.05	0.78	0.11	1.18	0.20	1.57	0.25	2.00	0.31	1.96	0.44	2.36	0.60	2.75	0.79	3.14	1.00	4.00	1.27	4.50	1.56	5.00
2	0.10	1.56	0.22	2.36	0.40	3.14	0.50	4.00	0.62	3.92	0.88	4.72	1.20	5.50	1.58	6.28	2.00	8.00	2.54	9.00	3.12	10.0
3	0.15	2.34	0.33	3.54	0.60	4.71	0.75	6.00	0.93	5.88	1.32	7.08	1.80	8.25	2.37	9.42	3.00	12.00	3.81	13.50	4.68	15.0
4	0.20	3.12	0.44	4.72	0.80	6.28	1.00	8.00	1.24	7.84	1.76	9.44	2.40	11.0	3.16	12.6	4.00	16.0	5.08	18.0	6.24	20.0
5	0.25	3.90	0.55	5.90	1.00	7.85	1.25	10.0	1.55	9.80	2.20	11.8	3.00	13.8	3.95	15.7	5.00	20.0	6.35	22.5	7.80	25.0
6	0.30	4.68	0.66	7.08	1.20	9.42	1.50	12.0	1.86	11.8	2.64	14.2	3.60	16.5	4.74	18.8	6.00	24.0	7.62	27.0	9.36	30.0
7	0.35	5.46	0.77	8.26	1.40	11.0	1.75	14.0	2.17	13.7	3.08	16.5	4.20	19.2	5.53	22.0	7.00	28.0	8.89	31.5	10.9	35.0
8	0.40	6.24	0.88	9.44	1.60	12.6	2.00	16.0	2.48	15.7	3.52	18.9	4.80	22.0	6.32	25.1	8.00	32.0	10.2	36.0	12.5	40.0
9	0.45	7.02	0.99	10.6	1.80	14.1	2.25	18.0	2.79	17.6	3.96	21.2	5.40	24.8	7.11	28.3	9.00	36.0	11.4	40.5	14.0	45.0
10	0.50	7.80	1.10	11.8	2.00	15.7	2.50	20.0	3.10	19.6	4.40	23.6	6.00	27.5	7.90	31.4	10.0	40.0	12.7	45.0	15.6	50.0
11	0.55	8.58	1.21	13.0	2.20	17.3	2.75	22.0	3.41	21.6	4.84	26.0	6.60	30.2	8.69	34.5	11.0	44.0	14.0	49.5	17.2	55.0
12	0.60	9.36	1.32	14.2	2.40	18.8	3.00	24.0	3.72	23.5	5.28	28.3	7.20	33.0	9.48	37.7	12.0	48.0	15.2	54.0	18.7	60.0
13	0.65	10.1	1.43	15.3	2.60	20.4	3.25	26.0	4.03	25.5	5.72	30.7	7.80	35.8	10.3	40.8	13.8	52.0	16.5	58.5	20.3	65.0
14	0.70	10.9	1.54	16.5	2.80	22.0	3.50	28.0	4.34	27.4	6.16	33.0	8.40	38.5	11.1	44.0	14.0	56.0	17.8	63.0	21.8	70.0
15	0.75	11.7	1.65	17.7	3.00	23.6	3.75	30.0	4.65	29.4	6.60	35.4	9.00	41.2	11.8	47.1	15.0	60.0	19.0	67.5	23.4	75.0
16	0.80	12.5	1.76	18.9	3.20	25.1	4.00	32.0	4.96	31.4	7.04	37.8	9.60	44.0	12.6	50.2	16.0	64.0	20.3	72.0	25.0	80.0
17	0.85	13.3	1.87	20.1	3.40	26.7	4.25	34.0	5.27	33.3	7.48	40.1	10.2	46.8	13.4	53.4	17.0	68.0	21.6	76.5	26.5	85.0
18	0.90	14.0	1.98	21.2	3.60	28.3	4.50	36.0	5.58	35.3	7.92	42.5	10.8	49.5	14.2	56.5	18.0	72.0	22.9	81.0	28.1	90.0
19	0.95	14.8	2.09	22.4	3.80	29.8	4.75	38.0	5.89	37.2	8.36	44.8	11.4	52.2	15.0	59.7	19.0	76.0	24.1	85.5	29.6	95.0
20	1.00	15.6	2.20	23.6	4.00	31.4	5.00	40.0	6.20	39.2	8.80	47.2	12.0	55.0	15.8	62.8	20.0	80.0	25.4	90.0	31.2	100.0
21	1.05	16.4	2.31	24.8	4.20	33.0	5.25	42.0	6.51	41.2	9.24	49.6	12.6	57.8	16.6	65.9	21.0	84.0	26.7	94.5	32.8	105.0
22	1.10	17.2	2.42	26.0	4.40	34.5	5.50	44.0	6.82	43.1	9.68	51.9	13.2	60.5	17.4	69.1	22.0	88.0	27.9	99.0	34.3	110.0
23	1.15	17.9	2.53	27.1	4.60	36.1	5.75	46.0	7.13	45.1	10.1	54.3	13.8	63.2	18.2	72.2	23.0	92.0	29.2	103.5	35.9	115.0
24	1.20	18.7	2.64	28.3	4.80	37.7	6.00	48.0	7.44	47.0	10.6	56.6	14.4	66.0	19.0	75.4	24.0	96.0	30.5	108.0	37.4	120.0
25	1.25	19.5	2.75	29.5	5.00	39.2	6.25	50.0	7.75	49.0	11.0	59.0	15.0	68.8	19.8	78.5	25.0	100.0	31.8	112.5	39.0	125.0
26	1.30	20.3	2.86	30.7	5.20	40.8	6.50	52.0	8.06	51.0	11.4	61.4	15.6	71.5	20.5	81.6	26.0	104.0	33.0	117.0	40.6	130.0
27	1.35	21.1	2.97	31.9	5.40	42.4	6.75	54.0	8.37	52.9	11.9	63.7	16.2	74.2	21.3	84.8	27.0	108.0	34.3	121.5	42.1	135.0
28	1.40	21.8	3.08	33.0	5.60	44.0	7.00	56.0	8.68	54.9	12.3	66.1	16.8	77.0	22.1	87.9	28.0	112.0	35.6	126.0	43.7	140.0
29	1.45	22.6	3.19	34.2	5.80	45.5	7.25	58.0	8.99	56.8	12.8	68.4	17.4	79.8	22.9	91.1	29.0	116.0	36.8	130.5	45.2	145.0
30	1.50	23.4	3.30	35.4	6.00	47.1	7.50	60.0	9.30	58.8	13.2	70.8	18.0	82.5	23.7	94.2	30.0	120.0	38.1	135.0	46.8	150.0

BETHLEHEM H PILES — PROPERTIES FOR DESIGNING



Section No.	Weight per ft.	Area of section	Depth of section	Flange		Web thickness	Axis XX			Axis YY		
				Width	Thickness		I	S	r	I'	S'	r'
	Lb.	In.^{2}	In.	In.	In.	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.
BP14	117	34.44	14.23	14.885	.805	.805	1228.5	172.6	5.97	443.1	59.5	3.59
BP14	102	30.01	14.03	14.784	.704	.704	1055.1	150.4	5.93	379.6	51.3	3.56
BP14	89	26.19	13.86	14.696	.616	.616	909.1	131.2	5.89	326.2	44.4	3.53
BP14	73	21.46	13.64	14.586	.506	.506	733.1	107.5	5.85	261.9	35.9	3.49
BP12	74	21.76	12.12	12.217	.607	.607	566.5	93.5	5.10	184.7	30.2	2.91
BP12	53	15.58	11.78	12.046	.436	.436	394.8	67.0	5.03	127.3	21.2	2.86
BP10	57	16.76	10.01	10.224	.564	.564	294.7	58.9	4.19	100.6	19.7	2.45
BP10	42	12.35	9.72	10.078	.418	.418	210.8	43.4	4.13	71.4	14.2	2.40
BP8	36	10.60	8.03	8.158	.446	.446	119.8	29.9	3.36	40.4	9.9	1.95

FIG. 4.

Notes: It is advisable to protect steel piles that are exposed, especially those that are subjected to the corrosive action of salt water. Fresh water may also be very injurious if it contains acids and other harmful chemicals; *e.g.*, ferric sulfate leached from piles of coal. Copper-bearing steel is advantageous when exposed to air but it seems to have little advantage in sea water.

Mortar is a better protection than concrete. The cement should be one of the special kinds that are made for use in sea water. The cover over any steel or reinforcement should be at least 4 in. The encasement should extend 3 ft. below the permanent mud line. Corrosion-resistant metallic shells used as permanent forms are very desirable.

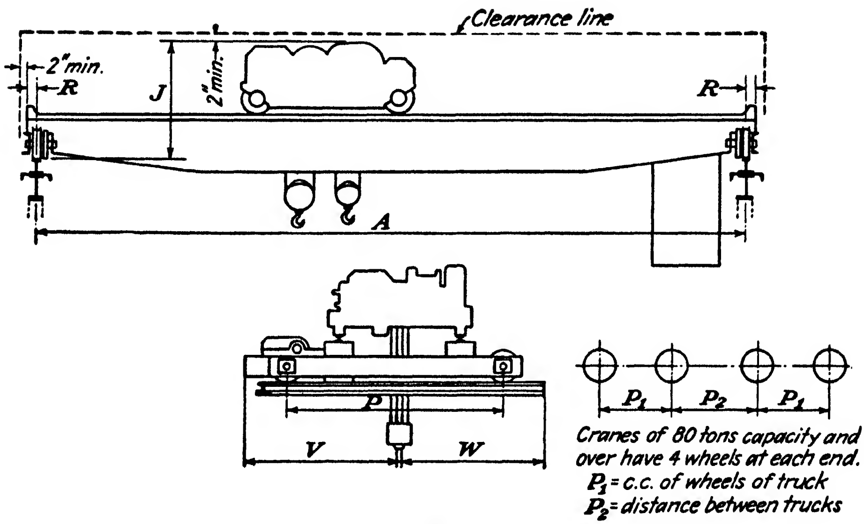
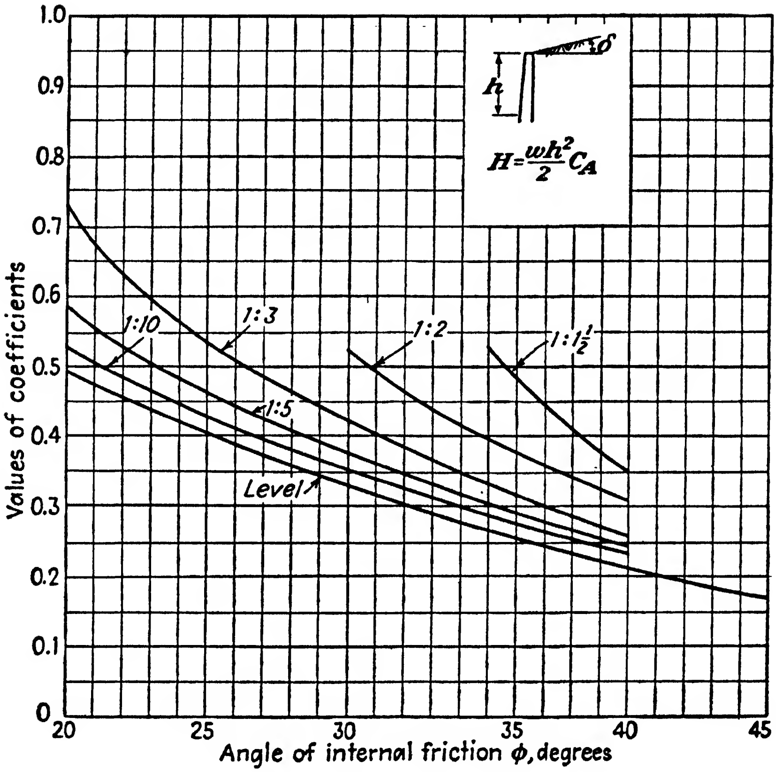


Diagram for Table 5.

TABLE 5. DIMENSIONS OF SHAW-BOX CRANES (SEE FIGURE ON OPPOSITE PAGE)

(Manning, Maxwell & Moore, Inc., Muskegon, Mich.)

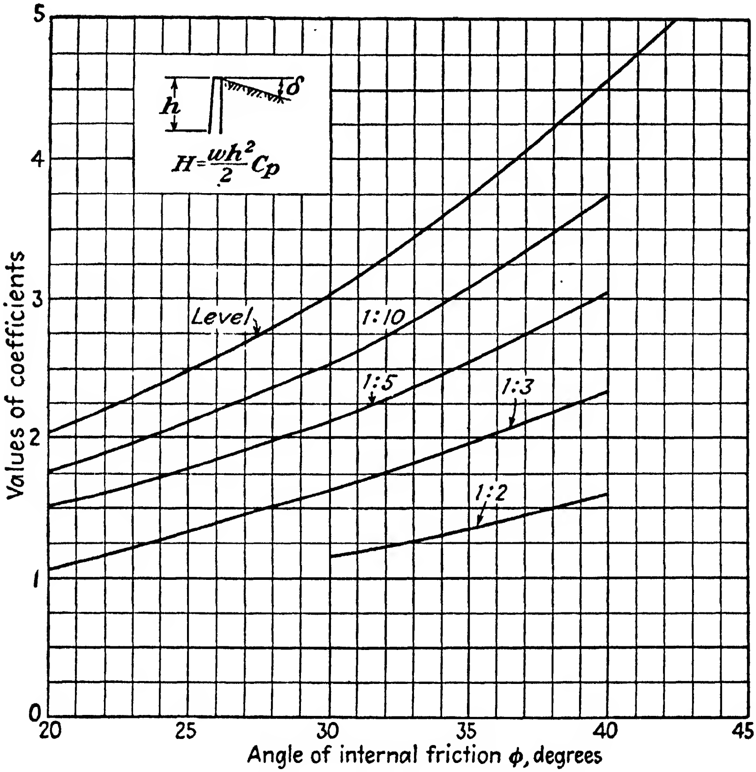
Capacity	A	J	P		R	V	W	Max. wheel load, lb.
10 tons	40′	5′1″	8′10″		7½″	6′9″	5′7″	19,000
	60′	5′7″	9′8″		8½″	7′6″	6′6″	24,000
	80′	5′7″	11′10″		8½″	8′6″	7′3″	28,000
	100′	6′0″	14′2″		8½″	9′10″	8′5″	35,000
20 tons (5-ton aux.)	40′	6′5″	11′0″		9″	8′7″	7′6″	37,000
	60′	6′8″	11′4″		10½″	9′1″	7′10″	42,000
	80′	6′8″	12′6″		10½″	9′5″	8′2″	47,000
	100′	6′8″	14′2″		10½″	9′10″	8′7″	54,000
30 tons (5-ton aux.)	40′	6′10″	11′8″		10¾″	9′2″	7′7″	50,000
	60′	6′10″	12′0″		10¾″	9′6″	7′10″	54,000
	80′	6′10″	13′2″		11½″	10′0″	8′6″	60,000
	100′	6′10″	14′6″		11½″	10′6″	9′2″	68,000
50 tons (10-ton aux.)	40′	8′4″	12′8″		13″	10′1″	8′7″	74,000
	60′	8′4″	14′0″		13″	10′3″	8′9″	81,000
	80′	8′4″	15′0″		14½″	10′11″	9′5″	92,000
	100′	8′4″	16′4″		14½″	11′8″	10′3″	100,000
			P_1	P_2				
80 tons (10-ton aux.)	60′	7′9″	4′8″	4′4″	12″	11′3″	9′7″	52,000
	80′	7′9″	5′0″	4′0″	12″	11′5″	9′10″	60,000
	90′	8′1″	5′2″	4′4″	12″	11′9″	10′2″	65,000
	100′	8′1″	5′6″	4′6″	12″	12′0″	10′6″	70,000
100 tons (15-ton aux.)	60′	8′0″	5′0″	4′6″	13″	11′6″	9′8″	78,000
	80′	8′6″	5′4″	4′2″	13″	11′8″	9′10″	86,000
	90′	8′6″	5′6″	4′6″	13″	12′0″	10′1″	80,000
	100′	8′6″	5′8″	4′4″	14″	12′0″	10′1″	95,000
150 tons (25-ton aux.)	60′	9′0″	5′0″	4′6″	14″	11′6″	9′8″	92,000
	80′	9′6″	5′4″	4′2″	14″	11′8″	9′10″	100,000
	100′	10′0″	5′8″	4′4″	14″	12′0″	10′1″	109,000
200 tons (25-ton aux.)	60′	9′6″	5′0″	4′6″	16″	11′6″	9′8″	109,000
	80′	10′0″	5′4″	4′8″	16″	11′11″	10′0″	119,000
	100′	11′0″	5′8″	4′4″	16″	12′0″	10′2″	130,000



Coefficients for Coulomb's equation for active earth pressure; horizontal component

$$C_A=\left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\delta)}}\right]^2$$

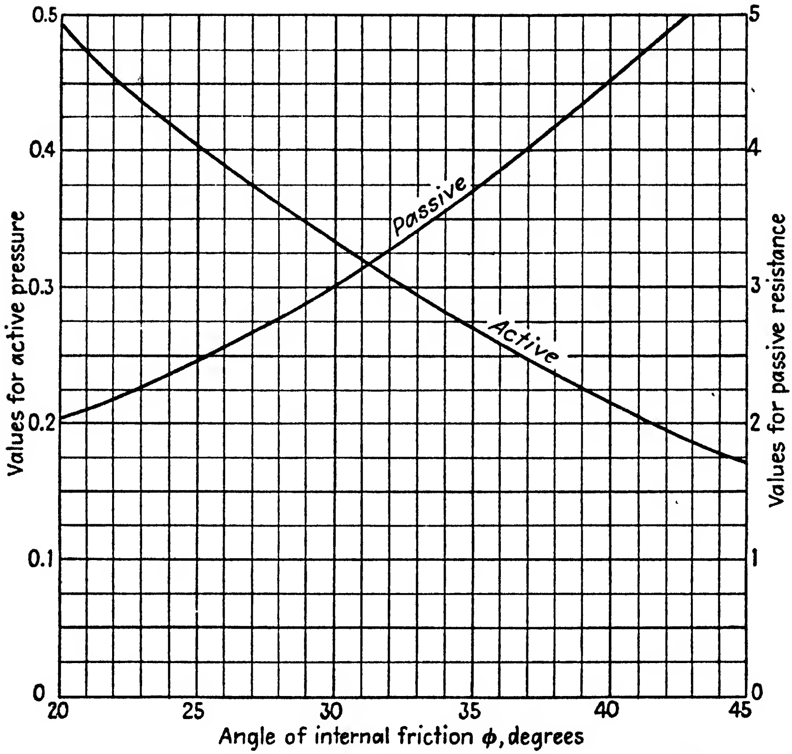
FIG. 5. Data regarding active earth pressure.



Coefficients for Coulomb's equation for passive earth pressure; horizontal component

$$C_p = \left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\delta)}}\right]^2$$

FIG. 6. Data regarding passive earth pressure.



Values of intensity of horizontal earth pressures for horizontal surfaces. Coefficients for wh in Rankine's formulas

$$\text{Active pressure} = wh\left(\frac{1-\sin\phi}{1+\sin\phi}\right) = wh\tan^2\left(45° - \frac{\phi}{2}\right)$$

$$\text{Passive pressure} = wh\left(\frac{1+\sin\phi}{1-\sin\phi}\right) = wh\tan^2\left(45° + \frac{\phi}{2}\right)$$

FIG. 7. Data regarding earth pressures as computed by Rankine's formulas.

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