



PART 4
DESIGN OF SHORELINE STRUCTURES



CHAPTER 20

THE INFLUENCE OF SUBSURFACE CONDITIONS ON THE DESIGN OF FOUNDATIONS FOR WATERFRONT STRUCTURES IN THE GREAT LAKES AREA

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The purpose of this paper is to describe subsurface conditions beneath the principal waterfront areas in the Great Lakes region and to discuss certain foundation problems associated with waterfront structures in this locality.

GEOLOGICAL SETTING

INTRODUCTION

During the slow and oscillating withdrawal of the continental ice sheets from the locality of the Great Lakes, various bodies of water existed in the basins now filled by the Great Lakes themselves. At different times each of the basins was occupied by larger lakes or by lakes standing at higher levels than at present. Therefore, the waterfront cities of the present time, now located along the shores of the existing bodies of water, lie almost without exception in areas once occupied by glacial lakes. This fact may be observed in Fig. 1, which shows the maximum extent of the former glacial lakes and the locations of selected cities to be discussed in more detail in this paper.

In some instances the glaciers deposited clayey tills directly from the ice onto the existing ground surface or onto the bottom of the glacial lakes. In other localities, or at other times in the same locality, the glaciers discharged their debris into the waters of the lakes, and the materials were sorted by sedimentation through the lake waters. Thus, most of the important waterfront cities on the Great Lakes are underlain by deposits of clay, some of which are directly of glacial origin, and others of which should be classed as lacustrine glacial clays.

Inasmuch as the lake levels occasionally stood nearly constant for considerable periods of time, the clay deposits are frequently covered by sandy deposits representing old beaches. In some of the Great Lake cities, the sand deposits are of great thickness and importance; in others they are absent or insignificant.

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During periods of exceptionally low water levels the drainage systems eroded substantial valleys into the terrain. If the lake levels then rose slowly, the mouths of these valleys often became filled with swamp vegetation mixed with silt brought in by the streams. Thus, valleys filled with soft organic deposits are often encountered in the clays. These filled valleys are sometimes covered by beach deposits, but their existence is often revealed by depressions in the existing land surface, and they are often approximately followed by the courses of present rivers. Since many of the Great Lakes cities are established at the mouths of rivers, many of the most important industrial locations are underlain by such buried valleys.

Thus, it is seen that the subsurface materials beneath the more important ports in the Great Lakes area are likely to consist of clay tills, of lacustrine clays, or of various organic shore deposits.

CONSISTENCY OF DEPOSITS

The principal problems associated with waterfront construction in the Great Lakes area arise from the presence of deep deposits of clay or organic materials. Until recently the consistency of such materials was described by such terms as very soft, soft, stiff, etc. These classifications were subject to misunderstanding and to various interpretations. Therefore, the ability to deal with foundation problems in the region depended to a great extent on the experience and personal judgment of those charged with the responsibility for the work.

Since the advent of soil mechanics about 30 years ago, the consistency of these materials has been determined quantitatively by means of a simple test known as the unconfined compression test. A cylindrical sample of the soil, in virtually intact state, is inserted in a testing machine and subjected to vertical load until it fails. The pressure on a horizontal cross section of the sample at the time of failure is designated as the unconfined compressive strength. A general impression of the meaning of the various numerical values for the unconfined compressive strength can be obtained from Table 1.

Numerous borings have been made in the region of the Great Lakes from which relatively undisturbed samples have been obtained and unconfined compressive strengths determined. Diagrams representing the results of borings typical of various important industrial regions are shown in Figs. 2, 3 and 4.

In the discussion of the boring logs, one should bear in mind that all the deposits adjacent to the Great Lakes are of extremely erratic character. Therefore, in one sense, there is no such thing as a typical boring. Nevertheless, each of the Great Lakes cities has subsurface characteristics that differentiate it from the others. The boring logs shown in Figs. 2 to 4 have been chosen to illustrate the most prevalent or characteristic conditions.

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Fig. 1. Location map of locations discussed.

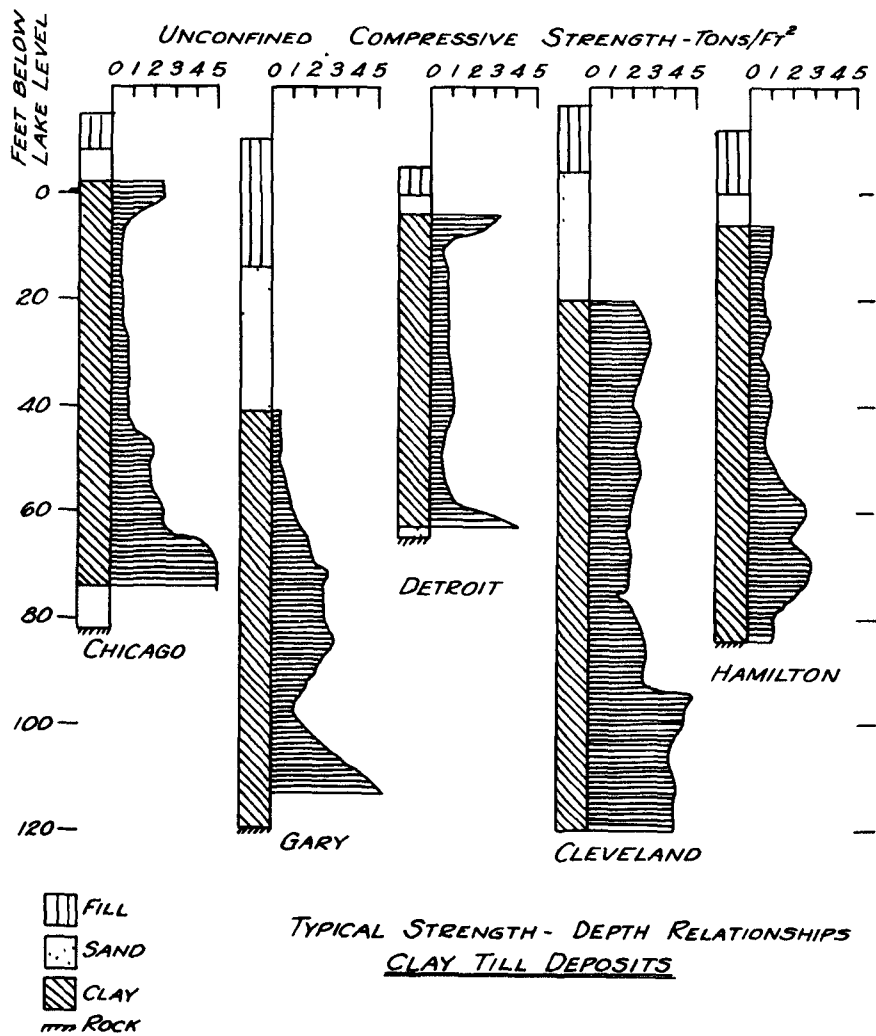


Fig. 2.

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Table 1. Qualitative and Quantitative Expressions
for Consistency of Clays

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq ft)
Very soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25-0.5
Medium	Can be penetrated several inches by thumb with moderate effort	0.5 -1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 -2.0
Very stiff	Readily indented by thumb-nail	2.0 -4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

CLAY TILL DEPOSITS

The localities of Chicago, Gary, Detroit, Cleveland and Hamilton are underlain primarily by glacial clay till. In Chicago, Fig. 2, there are found a few feet of fill, a few feet of sand, and a relatively deep deposit of clay. It may be seen that the upper 3 to 4 ft of the clay constitute a stiff crust having an unconfined compressive strength of about 2.5 tons per sq ft. The crust merges rapidly into relatively soft to medium clays having a strength not greater than 1 ton per sq ft for a depth of approximately 40 ft. With increasing depth, the stiffness of the clay increases in rather well defined steps. As a rule, each deposit of clay having a characteristic compressive strength represents one unique sheet of till associated with a specific advance and retreat of one of the ice sheets. At a depth greater than 60 ft the clay becomes very hard and may be underlain by granular materials. The bedrock consists of limestone. It is commonly encountered about 80 ft below lake level.

It is obvious that the principal difficulties with foundations in the locality of Chicago are the result of the 20 to 50 ft of soft and medium clays directly beneath the stiff crust. Under moderate load these clays are relatively compressible and cause large settlements. They are also responsible for large lateral pressures against bulkheads and wharves. Piles or piers find their support in the stronger underlying materials.

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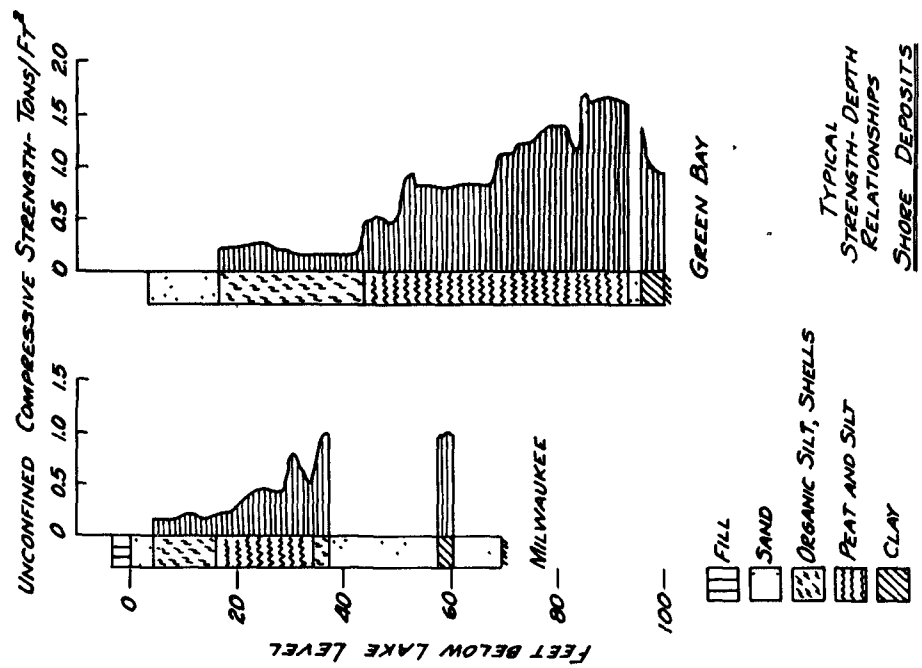


Fig. 4.

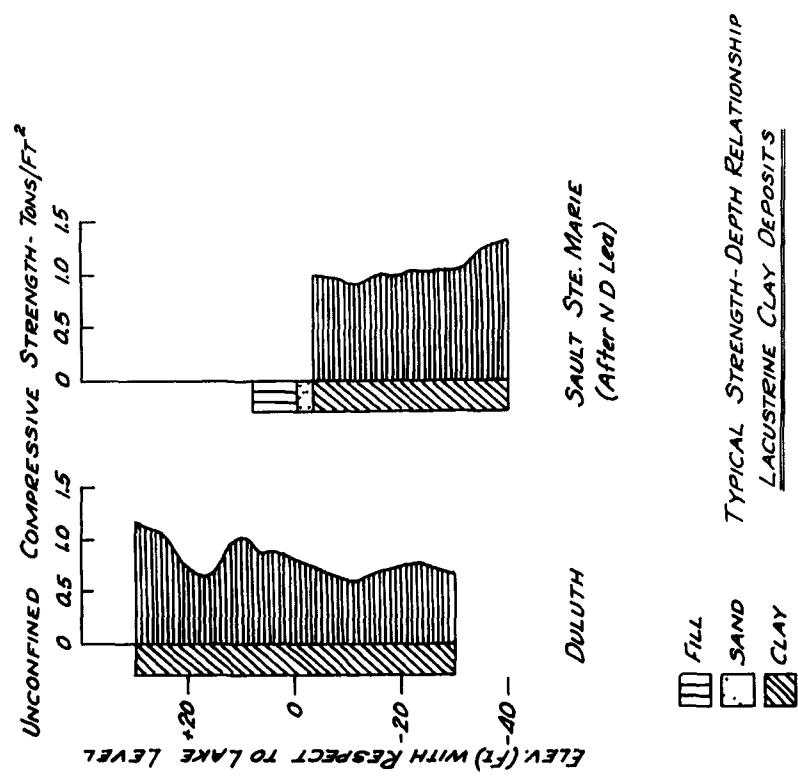


Fig. 3.

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The subsurface conditions in northern Indiana are exemplified by the boring log from Gary. The clays at Gary are somewhat similar to those in the Chicago area, but the surface of the clay is approximately 40 ft below lake level. Above the clay is found sand, or fill which itself usually consists of sand. The sand is relatively dense and forms a blanket capable of supporting most light structures. Beneath the sand is encountered clay which may be of soft to medium consistency, but which usually is stiff to very stiff. Only the heaviest and largest structures experience excessive settlement as a result of the presence of the underlying bodies of clay. Bedrock is somewhat deeper than in Chicago, generally about 120 ft below lake level.

The subsurface conditions in Detroit also have a general similarity to those in Chicago except for the fact that clays having an unconfined compressive strength less than 1 ton per sq ft extend almost the full distance from the clay surface to the bedrock. Therefore, waterfront structures in Detroit are usually more expensive to construct than those in the Chicago area, because the clays do not become stiffer with increasing depth.

In the main industrial section of Cleveland, located in the valley of the Cuyahoga River, the subsurface conditions are somewhat different from those previously described. Some 20 to 30 ft of sand and fill lie on top of the deposits of clay. However, at one time the sand was at least 100 ft thick and the underlying clays were subjected to much greater vertical pressures than they now experience. Therefore, they are all relatively stiff. The thickness of the clays is locally very great, because this part of Cleveland is underlain by a pre-glacial valley excavated into bedrock to depths as great as 600 ft below present lake level. It is obvious from a study of Fig. 2 that only the heaviest and largest structures in Cleveland should experience difficulty on account of the presence of the underlying clay.

The boring log characteristic of Hamilton, Ontario, shows many of the characteristics of that of Chicago and the subsurface conditions are quite similar.

LACUSTRINE CLAY DEPOSITS

The clays laid down in glacial lakes differ from the clay tills in that they do not contain stones or pebbles of large size, are likely to consist of laminations or bands of clay and silt, and are in general somewhat softer than the till deposits. Typical of these formations are borings from Duluth and Sault Ste. Marie, Fig. 3.

The boring at Duluth penetrated clays of medium consistency to a depth of about 50 ft, but the boring was not carried to the full depth of the clay deposits. It is probable that the compressive strength decreases still further as the depth increases. At Sault Ste. Marie the

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lacustrine clay appears to have an almost constant strength of about one ton per sq ft for a depth of about 40 ft. The clays at both localities are relatively compressible and are not blanketed by deposits of sand or stiff clay. Therefore, difficulties with waterfront structures are not uncommon.

SHORE DEPOSITS

The most significant characteristic of the various shore deposits or buried valleys is their extreme variability. The subsoil profiles are likely to contain sand, organic silt, shells, peat, and lacustrine clays in a completely heterogeneous array. The examples chosen, Fig. 4, from Milwaukee and Green Bay are illustrative of the great variability of the unconfined compressive strength of the organic silty deposits. It is seen that the compressive strength in both localities is at some levels somewhat less than 0.2 ton per sq ft. Such materials are incapable of supporting even the lightest structures and exert practically fluid pressures against bulkheads. Therefore, foundation problems are serious in both cities, and numerous examples of excessive settlement and of the lateral movement of retaining structures could be cited.

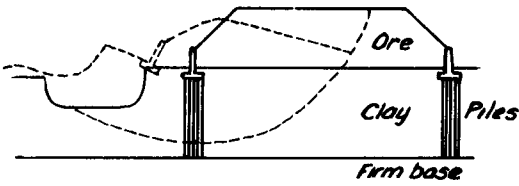
ORE STORAGE AREAS

INTRODUCTION

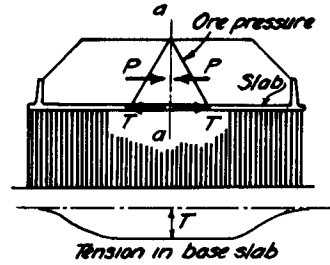
All standard types of waterfront construction are used along the shores of the Great Lakes. Most of these present no unusual features and do not deserve special discussion. In one respect, however, the waterfront facilities of the Great Lakes area are unique. Because the lakes are frozen for several months during the year, such heavy commodities as iron ore and limestone cannot be brought continually to the steel mills or other manufacturing centers where they are utilized. Therefore, storage spaces must be provided for these commodities in order to keep the plants in operation for a period of four to five months each year. Since space along the waterfront in the industrialized areas is at a premium, there is a natural desire to store such materials to the greatest possible height. At the same time, the unit weight of iron ore is relatively great (about 160 lbs per cu ft). Hence, the storage areas constitute large tracts of ground subjected to exceptionally high unit pressures. Many fields for the storage of iron ore are customarily piled to a height of at least 50 ft and exert a pressure of from 4 to 5 tons per sq ft over areas as great as 300 by 1000 ft. Since the storage areas are close to the waterfront, the lateral forces against dock structures may be very great. Therefore, it is not surprising that the areas for the storage of iron ore and for other heavy commodities have from time to time experienced large movements or even catastrophic failures. Several of these are described in the paper by Mr. E. J. Fucik.

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(a) BEARING-CAPACITY FAILURE



(b) PILE-SUPPORTED BASE SLAB



(c) CELLULAR SHEET-PILE CONSTRUCTION

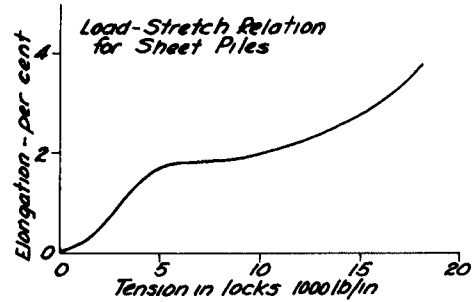
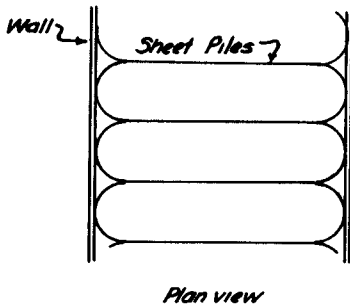


Fig. 5. General features of ore storage areas.

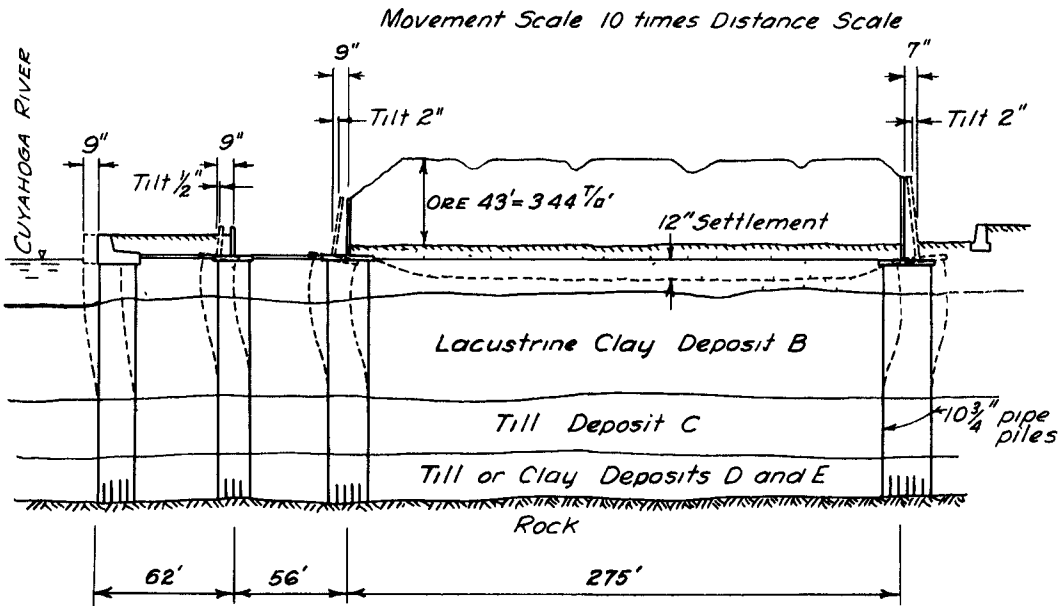


Fig. 6. Deformations of modern ore dock in Cleveland.

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The type of complete failure most commonly observed is that shown in Fig. 5a, wherein a large portion of the stored ore subsides, pushes the retaining structures laterally, and heaves the ground in front of the retaining structures in a vertical direction. If outright failure does not occur, large progressive movements may develop over a period of 20 to 30 years. Most of the ore docks in the Great Lakes region have experienced lateral movement on the order of several feet.

Methods of analysis have been developed for estimating the load that may be placed on the surface of a deposit of clay without incurring a failure such as that shown in Fig. 5a. These methods have been described by Terzaghi (1943). As a simple and crude approximation, however, the load that is likely to cause failure of a clay subsoil may be taken as approximately 2.57 times the unconfined compressive strength. This extremely simple rule indicates, for example, that iron ore could be piled only to a height of 16 ft before failure if the subsoil had an unconfined compressive strength of 0.5 tons per sq ft, whereas it could be piled to a height of 64 ft if the strength were as great as 2.0 tons per sq ft. In practice, of course, a suitable factor of safety is required.

CONVENTIONAL METHODS OF AVOIDING FAILURE

Several of the ore docks in the Great Lakes region have been supported by vertical piles capped by a reinforced concrete raft as shown in Fig. 5b. The piles have been driven to the firm base beneath the weak clays and have been designed to carry the entire vertical weight of the ore. The provision of such vertical support is often considered sufficient to insure the stability of the storage area. However, reference to Fig. 5b indicates that against any vertical section, such as a-a, there is a large lateral force representing the internal pressure built up within the ore itself. The magnitude P of this force can be computed with considerable accuracy by means of ordinary earth pressure theory. This force tends to cause the base of the ore pile to spread. If it is not resisted by adequate tensile reinforcement in the raft on top of the piles, the raft may split and the ore storage structure may fail by lateral movement. The tension T for which the slab must be reinforced is equal to the pressure P , per foot of length, exerted by the ore. Ordinarily an extremely high percentage of reinforcement is required to carry such loads.

The lateral ore pressures that cause the tension in the slab occur not only in the direction at right angles to the waterfront, but also in the longitudinal direction. Several ore storage areas have experienced failure or very large movements because the slabs were reinforced only in the direction at right angles to the dock and not longitudinally. One of the most recent examples of unsatisfactory behavior of an ore yard can be attributed to the lack of adequate longitudinal steel.

A pile-supported adequately reinforced slab constitutes a satisfactory structural solution to the ore storage problem, but is not likely to be attractive economically. Therefore, other procedures have often been advocated.

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SHEET-PILE DIAPHRAGM CONSTRUCTION

Several ore storage areas have been constructed by driving diaphragms of sheet piles transverse to the waterfront and tied together by semicircular arcs at the ends as shown in Fig. 5c. It is assumed that the vertical load on the clay within the sheet pile enclosures will produce lateral forces against the sheet piles which will tend to place each ring of sheet piles in tension. The tension in the sheet piles acts as a hoop tending to confine the enclosed clay and assures the stability of the structure. Construction of this type has occasionally been successful where the sheet pile cells were partly excavated and back-filled with granular material, but the method of construction has often been disappointing when no such precautions were taken.

One fallacy in the reasoning leading to the sheet-pile diaphragm construction consists in ignoring the fact that the sheet-pile walls must elongate appreciably in order to develop significant tension in the interlocks. Figure 5c contains a diagram showing for typical sheet piles the percentage elongation required to develop various amounts of tension in the interlock. To develop even a tension of 5000 lbs per lineal inch, a value that must be realized if the type of construction is to be at all economical, requires an elongation of about 2 per cent. Since the usual width of an ore storage structure is about 250 ft, the required extension of the sheet-pile diaphragm is on the order of 5 ft. Moreover, sheet-pile walls cannot be driven without a considerable amount of clearance in the interlocks of the piling. Such clearance probably amounts to at least 1/16 in. per interlock or a total of about 1 ft in 250 ft. Thus, before sheet-pile diaphragms could become effective in carrying a substantial part of the superimposed load, a spread of the base of the storage area of about 6 ft would be required. Movements of this magnitude are commonly considered excessive.

Furthermore, the ore stored in such areas is commonly placed in the form of pyramids which exert pressures longitudinally as well as transversely to the waterfront. A long transverse diaphragm affords no protection against longitudinal forces near its center. Therefore, it is quite possible that such diaphragms may be seriously displaced or ruptured in a direction at right angles to that for which they are designed.

Finally, it should be noted that the integrity of the sheet-pile diaphragm system depends upon the driving of every sheet-pile properly interlocked with its neighbor. A single pile driven out of lock renders an entire diaphragm useless. Where glacial tills are involved, the presence of pebbles and occasional boulders must be taken for granted. Therefore, an element of risk always exists in the driving of a continuous line of sheet piles. Furthermore, no method exists for detecting whether or not a pile has driven out of interlock. Therefore, the sheet-pile cellular construction, although extremely expensive, may not actually possess all the advantages that have been claimed for it.

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On some projects, the sheet-pile cells have not been extended entirely across the storage area, but have been restricted to a narrow retaining wall close to the waterfront. The cells in such a wall are often nearly circular in shape. Unless the clay in these cells is removed and replaced by granular material, the cells have negligible resistance to shearing deformations and do not serve their purpose. Indeed, even with granular fill, it is usually impracticable to satisfy the requirements of stability.

STRENGTHENING SUBSOIL BY CONTROLLED LOADING

The development of soil mechanics has led to the introduction of a basically different approach to the foundation problems of ore storage areas. It is well known that the application of a load to clay subsoils initiates a process of consolidation by which water is gradually squeezed from the pores of the clay. As this process continues, the strength of the clay increases appreciably. The increase occurs most rapidly when relatively large loads are placed on the clay. On the other hand, if the load applied to the clay produces large shearing forces in the subsoil, a failure may occur. Therefore, in order to achieve the maximum rate of increase of strength, a load only slightly less than the failure load should be applied, but great care must be exercised to make sure that a failure is not induced.

This principle may be utilized for the strengthening of the subsoils for ore storage areas, provided the quantity of ore to be stored in the first few years of the life of the facility can be restricted. Although it is undesirable to reduce the quantity of ore that may be stored from the point of view of operation of the blast furnaces, the cost of bringing in the required supplementary ore by rail may be substantially less than the cost of providing the additional initial strength by such artificial means as pile-supported rafts, etc.

Figure 6 shows a cross section of a modern ore dock constructed in Cleveland in accordance with the principle mentioned in the preceding paragraph. The retaining walls for the storage structure are established on steel piles to rock. No foundation was provided, however, for the ore storage area proper. Appreciable deformation of the structure was anticipated, and actually occurred. The deformations after a period of operation of four years are indicated in Fig. 6. It is seen that the walls moved out on the order of 9 in. The movements still continue but at a relatively slow rate. By controlling the rate of loading during the first four years the strength of the subsoil was built up to the point where virtually no restriction need now be placed upon the operation of the yard.

The process of strengthening of the foundation is illustrated in Fig. 7. The vertical pressure exerted by the ore on a strip 90 ft wide adjacent to one retaining wall is plotted as a function of the outward movement of the wall. During the first year of loading the average pressure increased from zero to approximately 2 tons per sq ft. During this

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period the deflection in the wall increased to about 4 in., and the shape of the load-deflection curve indicates that if the load had been carried appreciably beyond 2 tons per sq ft a failure would have undoubtedly occurred, because the curve was approaching a vertical tangent at about that value of loading. When the ore was used during the winter season the storage yard was almost completely unloaded. No appreciable recovery of the deflection took place. The second year of loading involved relatively small loads on the ore yard for operational reasons. Therefore, the second year of loading did not produce appreciable additional deflection of the wall. However, during the third year the load was increased to almost three tons per sq ft. The load-deflection curve indicates that the strength of the clay had been increased at least 50 per cent by the loading during the first two years. The load placed during the third year further increased the strength of the clay so that during the fourth year a pressure of about 3.3 tons per sq ft was applied with safety.

Figure 7 indicates forcefully and graphically that the strength of the clay beneath the storage area was nearly doubled in a period of four years by a process of increased loading carefully controlled to avoid failure. The avoidance of failure was guaranteed by a variety of types of field observations concerning the movements of the walls, the settlement of the clay surface, the excess water pressure developed in the pores of the clay, and detailed records of the position and amount of the various loads. Such control can hardly be attempted except under expert supervision, but once the strengthening has been achieved the structure is capable of carrying the desired loads with complete safety. Obviously, the cost of the construction is less than that of any of the types mentioned under the preceding subheading.

BEHAVIOR OF ANCHORED BULKHEADS

PRINCIPLES OF DESIGN

Numerous bulkheads consisting of vertical sheet piles tied back to anchorages have been constructed along all the Great Lakes industrial waterfronts. These have usually been designed in accordance with generally accepted procedures, and have for the most part performed in a satisfactory manner. Inasmuch as the latest information concerning the principles of design for anchored bulkheads has recently been summarized by Terzaghi (1953), no discussion of these principles will be attempted in the present paper.

However, it may be pointed out that experience indicates that the most frequent shortcoming of anchored bulkheads is the inadequate resistance of the soil in front of the buried part of the sheet-piles. The depth of the sheet piling is usually estimated on the basis of the assumed passive resistance of the soil in front of the bulkhead. If the strength of this soil is overestimated, failure of the bulkhead is extremely likely to occur.

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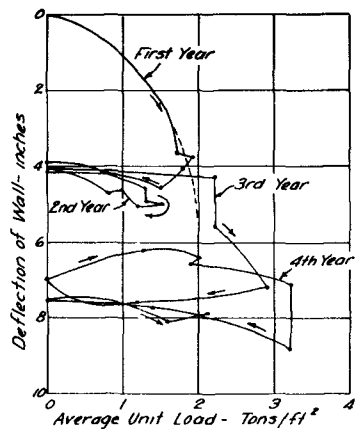


Fig. 7. Relation between wall deflection and load on 90-ft. strip adjacent to wall, Cleveland ore storage yard.

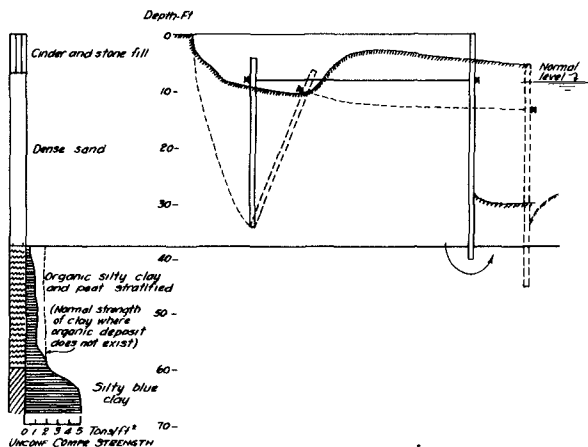


Fig. 8. Failure of sheet-pile bulkhead, Northern Indiana.

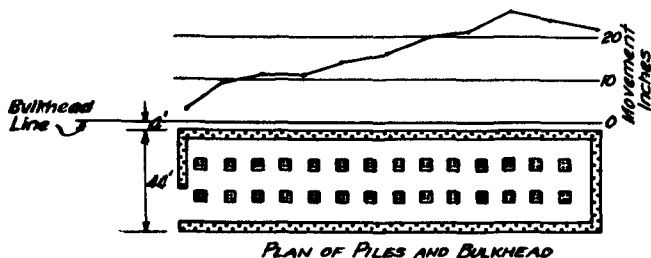
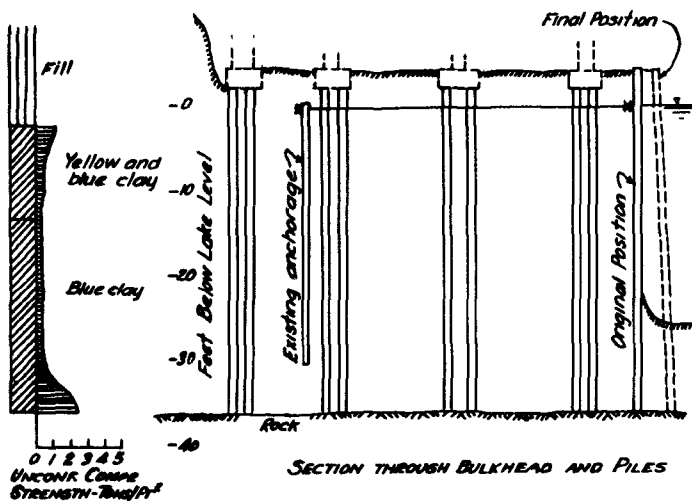


Fig. 9. Bulkhead movements caused by pile driving.

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EXAMPLE OF TYPICAL FAILURE

Figure 8 represents one of several bulkhead failures in northern Indiana. The design of the bulkheads is conventional, although not overly conservative. Failures have been uncommon, and have occurred over relatively short distances. The failure illustrated in Fig. 8 took place when a strong and steady wind from the south blew the water at the southern end of Lake Michigan to the north with sufficient energy to lower the water level in front of the bulkhead by about 3 ft in a few hours. At the period of the lowest water level, excessive movements occurred. From the character of the movements shown in Fig. 8 it is obvious that the toe resistance of the sheet piles was inadequate and that the sheeting moved out by translation almost without rotation. Ultimately, of course, the anchorage also proved inadequate, but it is doubtful that the anchorage would have failed had the toe resistance been great enough.

Borings indicated that the sheet piles were buried not in the clay normally found in the locality, with an unconfined compressive strength on the order of 2 tons per sq ft, but in a buried valley containing organic silty clay and peat with unconfined compressive strengths on the order of 0.4 to 0.6 ton per sq ft. The failure of the bulkhead was confined strictly to the locality where the organic deposits existed. Similarly, the extent of other failures in northern Indiana has been defined by the presence of organic deposits of soft consistency which replaced the stiffer clays normally encountered.

These examples indicate the necessity for at least a simple but a systematic soil survey to determine the toe resistance of the materials in which the sheet piles will be embedded. Where buried valleys are likely to be encountered, particular care must be taken to make sure that unanticipated soft spots will not be overlooked. The existence of such soft materials is a far more significant fact than the elaborate or precise determination of the physical properties of the normal soils if the occasional weak deposits are overlooked.

BUILDING FOUNDATIONS NEAR WATERFRONT

INTRODUCTION

The customary types of foundations are commonly utilized in the waterfront areas of the Great Lakes. Where structures would be inadequately supported by a crust of stiff clay or a blanket of dense sand, piles or piers are used to transfer the load to stiffer materials at greater depths.

Construction of pile-supported structures behind existing bulkheads, however, may be accompanied by substantial lateral movement of the bulkheads. This fact has often been ignored in the planning stages of substructures and

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has led to unpleasant surprises during the construction period. Therefore, this subject will be selected for somewhat more detailed discussion.

MOVEMENTS DUE TO PILE DRIVING

As an example that may be regarded as fairly typical, the lateral movements of a bulkhead due to the driving of piles for a building are illustrated in Fig. 9. The soil profile, including the variation in unconfined compressive strength, is shown in the diagram. It is seen that much of the structure is underlain by clay having an unconfined compressive strength of about 0.5 ton per sq ft, and that the clay is in a sense confined vertically by a blanket or crust of stiff yellow and blue clay.

During the driving of foundation piles it was observed that the bulkhead moved into the river as much as 2 ft at the top, and generally somewhat more at the bottom. The movement was progressive and tended to increase in the general direction in which pile driving took place.

The plan of the piles and bulkhead, together with a cross section through the foundation, indicates that the density of piles was not abnormally large. The average diameter of piles was approximately 1 ft. Nevertheless, the driving of these piles produced large and objectionable movements of the bulkhead and required expensive redesign of the foundation of the structure.

Such movements are most likely to occur when the piles are driven into saturated clays, and are less likely to occur if the subsoil consists of peats or organic silts. Although no strictly rational procedure can be suggested for determining the amount of movement to be expected during the driving of piles behind an existing retaining structure, the possibility of such movement should be kept in mind and may have a decisive effect upon the choice of foundation.

The importance of at least recognizing the possibility of such movements is attested by the fact that the cost of redesign and remedial measures has in numerous instances been far greater than the extra expense of installation of some nondisplacement type of foundation. If displacement piles are used the undesirable consequences may often be avoided or reduced by coring or other methods of preexcavating part of the material occupying the space in which the piles are to be located.

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- Terzaghi, K. (1953). Anchored Bulkheads. Proc. Am. Soc. Civil Engineers, Separate No. 262, Sept. 1953.