AN INTRODUCTION TO

DEEP FOUNDATIONS AND SHEET-PILING
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AND
SHEET-PILING
40286
BY
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Publisher's Note.—"Reinforced Concrete Piling" by the late F. E. Wentworth-Sheilds and the late W. S. Gray, and revised by H. W. Evans, deals fully with the design and manufacture of bearing piles and the construction of waterside piled structures.

For particulars of this
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See page facing 264

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PREFACE

In his earlier book, "Sheet Piling, Cofferdams and Caissons", the writer endeavoured to include in one volume a comprehensive modern treatment of many of the problems of design and practice implied by that title. The present publication incorporates that book, but the content has been extended and revised to cover most types of deep foundations.

As before, emphasis is placed on practical considerations influencing the choice of method or type of construction and on practical design. The opportunity is taken to include examples of more recent construction. An extensive bibliography is given so that the reader may pursue particular aspects in greater detail.

New matter includes a short preliminary consideration of soils, a brief review of recent developments in relation to bearing piles, some particulars, including design tables, of prestressed concrete piles, and under-water tunnels. The part dealing with working in compressed air has been improved and modernised with the kind of assistance of Sir Robert H. Davis, D.Sc. (Hon.), F.R.S.A., and Captain G. C. C. Damant, C.B.E., R.N. (Ret.).

The writer desires to acknowledge the helpful criticism of many friends, to tender his thanks to Mr. C. E. Riches, who assisted very materially with the original publication and with the present revisions, and to all those who have kindly lent illustrations or permitted quotations from their writings.

D. L.

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CHAPTER I

SOILS

The brief remarks in this chapter are intended only to be reminders of some matters relating to the properties of soils and site investigations.

It is assumed that the reader has a general knowledge of soil mechanics and methods of sub-soil investigation, but references (1.1), (1.2) to some of the many publications dealing with these subjects may be helpful to those who are not familiar with them. A general knowledge of geology is, of course, valuable or

![Diagram of soil classification triangle]

**Example.**—Clay having a grain-size analysis indicated by • contains 50 per cent. clay, 40 per cent. silt, and 10 per cent. sand.

**Fig. 1.**

even essential for engineers concerned with deep foundations, but the advice of a geologist familiar with the strata in the vicinity of a site is often also desirable.

Most soils are combinations of grains of a great range of sizes and various materials. It is seldom that clay does not also contain silt and often also sand. A method of classifying soils comprising one or more of these constituents is given in Fig. 1. Few soils lie on one of the three base-lines forming the sides

* References thus (1-1) refer to the Bibliography on page 9.
of the triangular diagram; most lie inside the triangle. Most soils contain little organic material; peat is an exception. Since organic material can affect greatly the physical properties of a soil, a test for loss on ignition, or in the case of a sandy soil, a simple comparative test such as that described in B.S. No. 812 is usually desirable.

It is important to foresee possible changes in the properties of a soil, for example due to seasonal changes, and also changes that may arise from constructional work likely to be undertaken. In the design of a foundation, provision should be made both for soil conditions during construction and for possible exceptional conditions in the future.

With clay and silt, effective drainage may be of great assistance in increasing the passive resistance and preventing high active lateral pressures.

![Diagram showing classification of soils](attachment:soil_classification_diagram.png)

**Fig. 2.—Limit Sizes of Grains for Various Methods of Dewatering.**

Sieve tests as described in B.S. No. 812 are used to determine the proportions of particles larger than fine sand. Particle-size classification of silt and colloidal materials is determined by a hydrometer test described in B.S. No. 1377 and ASTM. D422.

*Fig. 2* shows the range of sizes of grains of sand, silt and gravel, and also the ranges of distribution of grains of different sizes for soils that can be dewatered by various methods.\(^{(1,2)}\)

A soil that is mainly silt can be identified on the site by a wet sample exuding water if squeezed in the hand but re-absorbing the water when released. A soil having more than a small clay content can be identified by the stain left on the hand from a wet sample.

**Physical Properties of Soils.**

All soils with a measurable content of clay or silt exhibit variable resistance to shearing according to the water content, and shrink on drying. Soils without any clay or silt, apart from rock, do not suffer major change in physical properties between the inundated and the dry state, but sand, particularly fine sand, may become "quick", if subjected to an upward flow of water.

**Cohesion.**—Many finer granular soils have cohesive properties when damp but not when dry or immersed. Clay having a water content above the plastic
SOILS

limit but below the liquid limit is in its familiar state of plasticity. The plastic range of sandy soils is very small. For silt the liquid limit is about 35, and the plastic limit is about 20, giving a range, or plasticity index, of 15. For clay the plasticity index is seldom less than 25, and for bentonite much more.

Resistance to shearing of a non-granular soil due to a load on the surface, if no penetration of the soil by the load occurs, is due solely to the cohesive strength.

Colloids.—It is the presence of colloids which gives to clays their cohesive strength and other distinctive physical properties. It has been stated that this is due solely to the extremely small size of the grains; this is a generalisation.

The structure of clay has been assumed to be as shown in Fig. 3, but this is still the subject of research. If the surface-tension of the particles is reduced, for example by sodium oxalate, the clay will be deflocculated and the time for settling of suspended particles enables the grain-size distribution to be determined. The opposite action occurs when particles of soil in suspension in a river flocculate and settle relatively quickly when meeting salt water in the sea. When a soil is deflocculated the surface-tension of the particles is reduced. Apart from the agents mentioned in the foregoing this action may be caused by an electrolyte, such as a solution of an alkali, and by some organic liquids, for example sodium carbonate. The soil may be flocculated by an electric current which decomposes a salt and releases adsorbed positive ions, and also by a colloid of sign opposite to that of the clay, for example, ferric-hydroxide, or by an acid or a salt having ions which are readily adsorbed.

The constituents of most sands, silts and clay are silica or are derived from felspar. Therefore the assumption of a uniform specific gravity is seldom far wrong, since heavy minerals, like barytes, seldom exist in the form of small grains and can be detected by panning. Settlement of soil shaken up in the water follows Stokes's law. Assuming the organic matter is removed by hydrogen-peroxide and dilute hydrochloric acid followed by deflocculation, any particles in suspension near the surface after the liquid has been allowed to stand for one minute at a temperature of 20 deg. C. is silt or clay. Soil still in suspension after 4 hours is clay.

The voids in clay are entirely filled with water. Therefore clay is incompressible to impact loading and driving piles into clay causes the surface of the ground to rise or causes spreading laterally, or both effects may occur. The

Fig. 3.—Flocculated and De-Flocculated Particles.
volumes of these displacements are together equal to the volume of the pile embedded in the clay. However, applying a load on clay causes consolidation in course of time at a rate dependent on the existence of permeable strata above or below or of a free surface. Structures on the surface generally cause bulbs of pressure in the subsoil, the maximum intensity being under the centre of the load. In the case of clay this causes some redistribution of weight of the structure towards the outer edges with possible considerable increases of stress in a rigid structure, or a tendency for dishing settlement to take place in a non-rigid structure.

The shrinking of silts and clays, and soils containing these materials, due to reduction in the water-content, are too variable to tabulate in any simple way. However, it has been shown that the shrinking of cohesive soils is closely related to the plasticity index \(^{(1.10)}\) and this facilitates testing.

Mutual attraction between particles of soils with no clay content is either negligible or, in the case of silt, readily lost with the small increase of the water-content which causes the familiar treacherous behaviour of silt. In the case of clay, however, when the water-content is below the liquid limit, the mutual attraction gives the soil its cohesive strength, the ability to resist a load without the surface being depressed locally and also enables a vertical exposed face to stand without support. The cohesive strength is, however, approximately inversely proportional to the water-content, so necessitating design to be based on the assumed highest water-content probable during and after the constructional work.

Increase of Bearing Capacity with Depth.

A granular soil has no shearing resistance at a free surface but it increases fairly rapidly with increase of depth because of the appreciable internal friction. Since the shearing strength of cohesive soils usually derives almost entirely from the cohesive strength, and very little or none from internal friction, the strength of a cohesive soil of constant water-content increases very little with increase of depth.

Experimental verification of the variation of vertical pressure with depth generally shows that the actual pressures at some depth below the surface exceed slightly the theoretical pressures, such as those calculated by Boussinesq’s formula. It must not be overlooked, however, that the pressure in the soil close under the loaded area is considerably affected by the relative stiffnesses of the foundation, or other member applying the load, and the soil. The distribution of pressure on a loaded area at the surface is parabolic for granular soils but greater at the edge than in the centre for cohesive soils. These distributions are often wrongly assumed to also apply below the surface, but as the depth increases the pressure becomes more uniform on clay and sand.

In the case of shallow foundations, seasonal reduction in the water-content of the soil will increase the strength of the soil, but settlement is caused by the shrinking. For foundations at any depth, settlement due to increased pressure on a cohesive soil is due solely to expulsion of water from the soil until a new equilibrium is reached, at which stage the particles of soil alone are again carrying the load. The usual long time for this action to be completed is due to the low permeability of cohesive soils, and also to the thickness of the stratum and the permeability of the strata above and below. Therefore settlement takes place
much more quickly in strata of clay with intervening veins of sand or other permeable material.

Most failures of foundations are associated with silt and clay. Some failures result from confusing these two types of soils, but the majority are due to overestimating the shearing strength and from under-estimating the amount of settlement in course of time.

Analyses of soil samples and testing the properties of those expected to be suitable for loading is now usually carried out in a laboratory, but rough deter-

![Graph showing Bell's Test for Clay](image)

**SAFE BEARING CAPACITY FOR STRIP FOUNDATIONS BASED ON FAILURE BY SHEARING (FACTOR OF SAFETY: 3).**

- Sandy clay: $\phi = 4$ deg. $2.3 \times$ cohesion (lb. per sq. in.)
- Clay: $\phi = 0$ deg. $1.9 \times$ cohesion (,, ,, ,, ,)

For square foundations, increase coefficients by 20 per cent.

**Fig. 4.** (See also page 65.)

![Graph showing Bell's Test for Clay](image)

minations of the cohesive strength of newly-exposed clay may be made on a site by Bell's method,\(^{(1-11)}\) utilising a steel ball of $1\frac{1}{2}$ in. diameter in conjunction with the diagram in Fig. 4. Thus for a 24-in. drop, if the width of the depression is 0.9 in., the cohesive strength is 15 lb. per square inch. For a square foundation on a pure clay soil, the safe bearing pressure is therefore $1.2 \times 2.3 \times 15 = 41.4$ lb. per square inch $= 2.7$ tons per square foot.

**Subsoil Investigation.**

Investigation of the subsoil is necessary for all civil engineering works, and can be carried out in a number of ways. Should geological surveys be available and the ground is rock, it is nevertheless sometimes necessary to bore to appreciable depths to prove the accuracy of the information available and especially
Fig. 5.—Boring Equipment.
to determine if faults, permeable strata or soil of low shear strength exist. Borings are particularly necessary where faults or other discontinuities may limit the ability of the ground to support safely vertical or inclined forces acting downwards, or to resist uplift from anchorages (as in the case of prestressing of dams).

**Boring in Rock.—**Where hard rock has to be drilled and cores obtained for subsequent examination at the surface, diamond drilling is often used, and although expensive, this is the usual method of obtaining accurate information about hard strata in the form of continuous cores. Diamond drilling can be carried out to great depths, and a boring-rig for such an operation is shown in Fig. 5a. For drilling to less depths, mobile rigs such as those shown in Figs. 5c and d are used. In the former, the rig is mounted on a lorry and the mast can be laid in a horizontal position during travelling; this equipment is also suitable for taking samples of softer soils. The equipment in Fig. 5d is similar to that in c but is mounted on a skid.

Bore-holes greater than about 5 in. in diameter and from which cores can also be recovered are sunk by Calyx drills in which chilled steel-shot is the abrasive. Holes up to more than 4 ft. diameter are possible but small holes only are usually needed for cores for sub-soil investigation. This method is restricted to holes that are vertical or only slightly inclined.

**Churn Drilling.—**If cores are not needed for examination at the surface or for testing the soils penetrated can be identified by ordinary well-boring, or churn-drilling, methods. The penetration is caused by the blows of a hard-face bit which is rotated slightly at each blow and which breaks up the rock. The debris is formed into a thick slurry by feeding water into the hole. This method is not particularly suitable for sub-soil exploration except to determine the levels of definite changes in the strata, but is much used for sinking holes of small diameter preparatory to blasting and for wells up to several hundred feet deep. Shallow holes for blasting or for proving the thickness of a rock stratum are most conveniently sunk by air-driven percussion drills with tungsten-carbide bits.\(^{(1,12)}\)

**Auger Boring.—**Boring by means of an auger, which is usually mounted on a lorry, is not suitable for hard rock but is ideal for clay or marl and is reputed to be suitable for soft rock. For sub-soil exploration holes of 5 in. diameter, or slightly greater, enable undisturbed samples of soil to be taken for testing by interrupting the boring or for vane shearing tests to be made. Augers of large diameter for boring holes for piles are described on page 19 (Chapter II). A long auger, called a continuous-flight auger, is shown in Fig. 5b.

**Undisturbed Samples of Soil.**

To obtain undisturbed samples of about 4 in. diameter of cohesive soils from the depth at which it is required to know the properties of the soil, a tube is driven into the soil at the bottom of the boring. In this type of sampling, water is not used for the removal of the soil, or at least not for those parts of the bore where undisturbed samples are required. Methods of making soil investigations of this type, and the methods of testing are described in many books.\(^{(1,13), (1,14), (1,15)}\)

For greater convenience, and particularly on a large site where a large number of tests are desirable, undisturbed samples of 1½ in. diameter are usual, and are obtained by forcing the sampling tube into the clay at the bottom of a larger bore. This can be done intermittently as the penetration of the borehole is
increased. The tests to determine the shearing strength can be made without delay on the site with apparatus of the type shown by Fig. 6. Otherwise the samples must be protected from drying by coating them with wax and must be sent to a laboratory for testing. If soil investigations are to be carried out under water, it is usual for the tube lining the bore to extend above the level of the water.
Other Methods of Investigation.

Preplving Surveys.—With soft soils having a limited load-bearing capacity, or which may be liable to scour in the beds of rivers, and where general information about the sub-soil exists and where piling or other deep foundations are known to be necessary, a suitable method of determining the load-bearing capacity is to drive small piles such as steel joists, tubes, or similar, and record the penetration (or set) for each blow of known impact energy. This method enables the resistance to penetration of the soil to be determined as the penetration proceeds. If an enlarged foot is provided, resistance at the bottom of the pile can be recorded excluding the friction on the sides. This method does not give any other precise information about the soil, and is, therefore, more suitable for cases where the general information about the soil is known.

Geophysical Prospecting.—If the approximate depth of rock, or of water-bearing strata, below the surface is required to be known, use can be made of one of the geophysical methods of soil exploration. By the electrical-resistivity method, the large variations in conductivity of different types of soil enables the depth to a particular stratum to be deduced by recordings made from two points on the surface, the observations being repeated over the site. The line of an underground water-course or a metal pipe can usually also be found in this way. By the seismic method the times of arrival, at two points, of the reflected wave from small earth shocks caused by an explosion enable the depth of the top of rock to be determined and other information also obtained.

Marine Soundings.—The depth of water in rivers and the open sea is readily obtained by echo soundings from a boat; it is important, however, to relate these depths to the level of the water at the time the observations are made.

BIBLIOGRAPHICAL REFERENCES (CHAPTER I).

1.5—"Methods of Test for Soil Classification and Compaction." British Standard No. 1377.
1.8—"Engineering Properties of Soils." By C. A. Hogentogler. 1937.
1.17—"Introduction to Geophysical Prospecting." By M. B. Dobrin. 1952.
CHAPTER II

BEARING PILES

The most common forms of deep foundation are ordinary bearing piles, cylinders and caissons (see Chapters IX and X). The various types of bearing piles are (a) pre-formed piles driven by impact, for example precast concrete piles, timber piles, and steel piles of rolled H-section or welded box sections; (b) hollow cylinders driven by a steel mandrel (for example precast concrete or corrugated or plain steel tubes) which are subsequently filled with concrete; (c) tubes driven with the lower end closed, the closure left in place, and the hole in the ground filled with concrete as the tube is withdrawn; (d) ordinary bored piles up to about 8 in. diameter for which a hole bored in the ground is subsequently filled with concrete; (e) screw piles; and (f) forming a hole upwards of 8 in. in diameter by means of an auger, or a steel tube sunk into the ground by impact or by a combination of weight and reciprocating angular rotation, the hole being subsequently filled with concrete.

Resistance to Penetration of Driven Piles.

The method of calculating the safe load on a pile driven by impact has for many years been a controversial subject, but recently more data from loading tests have become available for comparison. The calculation of the greatest load which a bearing pile will support should be based essentially on the physical properties of the strata below the bottom of the pile. The properties should preferably be obtained from tests of samples of the soil. The compressibility of the soil below the bottom of the pile is an important factor, particularly in the case of cohesive soils, the moisture content of which will be reduced, sometimes over a long period, by the increase of the pressure on the soil. Due to overlapping of the bulbs of pressure below the bottom of piles in a group the pressure on the soil below the piles may greatly exceed the pressure under an isolated pile. It is necessary to know the properties of the soil at a depth equal to one and a half times the depth to which it is expected the piles will be driven, or even deeper if strata of soft cohesive soils are likely to occur within that part of the bulb of pressure in which the pressure on the soil is increased by 20 per cent. or more. A static formula should be used for estimating the necessary length and safe load of a pile, but for driven piles an impact formula may be used as a check and to ensure that each pile in a group has practically the same safe load and that the settlement is small and is practically equal for all the piles. If piles have to be driven when the condition of the ground cannot be known in advance or obtaining undisturbed samples of the soil is difficult or impossible, dependence must be placed on an impact formula from which the safe load is estimated from the resistance to penetration of the pile as measured by the set of the pile for one or more blows.

In the light of recent increased knowledge of this subject, the writer considers that most of the early formulas should be disregarded. Hiley's formula,\(^{(2.1, 2.3)}\) which has been extensively used in Great Britain and the Common-

* References thus \((2.1)\) refer to the Bibliography on page 28.
wealth, and with variations by Mr. J. S. Crandall (2.3) also in the U.S.A., is considered, in its original form, to give results too low for long piles. The Hiley formula estimates the resistance to penetration at the time of driving and not the ultimate static load, which is defined by Terzaghi as the least load causing a settlement equal to a tenth of the diameter (or width) of the pile. Therefore an allowance must be made for the reduction in the permissible load in the course of time if cohesive soils exist below the bottom of the pile. However, an addition can be made for the "take up", or increase of resistance, if applicable, after driving has ceased. The same adjustments apply to the resistance to penetration if calculated by the stress-wave theory, but Bullen’s formula (2.4) is related to the minimum static load causing permanent penetration, with a plasticity factor for use for piles in cohesive soil.

The stress-wave theory was first suggested for application to driven piles by Mr. D. V. Isaacs and subsequently was used by Dr. E. N. Fox in assisting Dr. (now Sir) William Glanville (2.5) and others in determining the instantaneous stresses occurring in a concrete pile while being driven for comparison with piezo-electric recordings of the stresses in the pile. It was not suggested by Glanville and others that the resistance to penetration during driving could be deduced from the stresses. In subsequent research by Mr. E. A. Smith (2.6) it has been assumed that, if some allowance is made for damping, the theory is likely to lead to much better results than given by any impact formula.

Faber, (2.7) ignoring all existing formulae and the stress-wave theory, related the energy of the impact to the results of loading tests on piles and deduced formulae for granular and for cohesive soils. The formulae (see page 12) forecast the ultimate static load of the pile instead of the resistance to penetration during driving. The greatest load obtained by the loading tests was not related to an exact amount of permissible settlement, and the results could not be expected to be exact, and were not certain to apply to long piles. Nevertheless the formulae can in general be considered a fair and generally conservative estimate of the ultimate load of an isolated pile.

Bullen’s formula (see page 13) was proposed following an investigation of the energy absorbed in the packing on the head of a pile during driving, the elasticity of the pile, and the elastic quake of the soil due to the impact. These factors correspond to the terms $C_h$, $C_p$ and $C_s$ in the Hiley formula and $C_1$, $C_2$, and $C_3$ in Bullen’s paper. In comparing test loads with the results obtained by his formula, Bullen adopted Terzaghi’s definition of “ultimate load” as given in the foregoing. Close agreement was obtained, but as the tests did not deal with long piles adequately some modification of the formula may yet be desirable.

Results obtained by E. A. Smith, based on the stress-wave theory, show generally greater resistance, particularly for long piles, than do Bullen’s or Faber’s formulae.

Results by the stress-wave theory show that the general form of resistance to penetration follows that of the older Wellington (or Engineering-News) formula, and does not conform to the results obtained from those formulae which contain a term representing the efficiency of the blow, for example, the formulæ of Redtenbacher, Hiley, and Janbu. It is possible that the stress-wave theory can be adapted to give the resistance to penetration very closely, but it is necessary to make allowances, which may sometimes be considerable, for the reduction
in ultimate static load where the pile is in cohesive soil or where cohesive soil exists at a lower level but still within the bulb of pressure. Pending further experience, it is probable that, for concrete piles, the results obtained by the stress-wave theory, as currently presented, will exceed the resistance to penetration due to a static load applied for a short time, and still more that due to a permanent load, on piles bearing on strata of cohesive soils, depending on the increase in the pressure on the subsoil and the properties of the soil. E. A. Smith has applied a damping factor, and Dr. D. E. Jenkins, in assisting the writer, has found that an allowance for the inertia of the pile will account for a reduction of 30 per cent. to obtain the static resistance to penetration. This factor is used in the following.

**Notation.**

The signification of the symbols in the formula for impact-driven piles and for the stress-wave theory is as follows.

- $A$, Cross-sectional area of pile.
- $b$, Breadth of pile.
- $C_1$, Temporary compression of the dolly and head packing.
- $C_2$, Temporary compression of the pile.
- $C_3$, Temporary elastic compression of the ground.
- $d$, Depth or side of the pile.
- $E_c$, Modulus of elasticity of the concrete.
- $e$, Equivalent elastic set.
- $f$, Frictional resistance on surface of pile.
- $h$, Effective free fall of hammer.
- $h_1$, Effective free fall of hammer allowing for inertia of the helmet.
- $h_r$, Fall of hammer on pile driven to refusal.
- $l$, Length of pile.
- $M$, Weight of hammer.
- $m$, Weight of helmet.
- $R$, Resistance to penetration (static load).
- $s$, Set per blow.
- $W$, Total weight of pile.
- $w$, Weight of pile per foot.

A method of recording the set $s$ and the quake $C_3$ of the ground is shown in Fig. 7, in which $e = C_3$.

**Formulae for Impact-driven Piles.**

The formulae due to Faber and Bullen are as follows.

**Faber’s Formulae.**—These formulae estimate resistance to penetration of a concrete pile by means of a static load $R$.

For granular soil:  
$$R = \frac{M(h - \frac{d}{z})}{s + x_1h}.$$  

For cohesive soil:  
$$R = \frac{M(h - \frac{d}{z})}{s + \rho x_1h}.$$  

The value of the coefficient $z$ is about $7$; the value of $\rho$ may usually be
taken as 4. For both types of soil, \( x_1 = \frac{2}{3} \frac{1}{\sqrt[3]{\frac{3d}{2}} \sqrt[7]{\frac{Ml}{AE_c}}} \) and is generally about 0.02. The factor of safety recommended by Faber is 1\(\frac{1}{2}\) to 2\(\frac{1}{2}\), depending on the engineer's assessment in a particular case.

![Diagram of a pile and measuring device]

**Fig. 7.—Measuring the Quake of the Ground.**

**Bullen's Formula.**—This formula also expresses the resistance to penetration of a concrete pile by means of a static load \( R \).

\[
R = \frac{Mh}{s + C_3 + 0.707xh} + 2(b + d)fL.
\]

When a pile is driven to refusal \((s = 0)\) and the free fall of the hammer is \( h_r \),

\[
R = \frac{Mh_r}{C_3 + 0.707xh_r} + 2(b + d)fL.
\]

The value of \( x \) is about 0.08 or less. The second term in the expression represents the frictional resistance of the sides of the pile. Bullen suggests, as a result of pull-out tests, a maximum frictional stress \( f \) of 0.37 tons per square foot, but that this stress is generally between 0.15 and 0.25 ton per square foot.

For piles in cohesive soil, Bullen considers the total resistance to static load is some proportion of that given by the basic formula and is related to the reduced quake of the soil \( \frac{C_3}{(M + m - 0.3W)} \). The proportion varies from 25 per cent. for a pile in fissured clay to about 80 per cent. These values apply to an isolated pile and lower values must be used for piles in a group or cluster.
Application of the Stress-wave Theory.

Since the determination of the resistance to penetration by the stress-wave theory is too laborious for general use, the following method, developed by the writer, may be applied to concrete piles more than 35 ft. long. The method, which has been adapted from the work of Glanville, is based on the need to limit the greatest stress at the top and bottom of the pile so that the pile will not be damaged. An instantaneous maximum stress of 2500 lb. per square inch has been used by the writer as the desirable instantaneous stress for concrete having a cube-strength of 5000 lb. per square inch, and should provide a reasonable factor of safety.

The determination of the fall or drop of the hammer and the stiffness of the packing on the top of the pile, which factors control the stress at the top,
and the least equivalent elastic set, which controls the stress at the bottom of
the pile, can be achieved in one graphical operation by use of the chart (Fig. 8).

E. A. Smith has shown that the reduction of stress in the length of the pile
by propagation, apart from friction on the side, is almost negligible, so that
the approximate resistance to penetration, including the friction on the side, is
given by the area of the cross-section of the pile multiplied by the stress of 2500 lb.
per square inch at the top multiplied by a factor to allow for the inertia of the
pile. It is usual to include the friction on the side with the bearing resistance
under the foot of the pile, and the friction tends to be increased by the " take-up "
on cessation of driving. However, in the case of a pile in filling which may settle
in course of time, the resistance to penetration will decrease because the settling
of the ground around the pile tends to drag the pile downwards by the friction
acting in the reverse direction.

The stress-wave theory emphasises the importance of the stiffness of the
packing, or cushion, under the helmet on the top of the pile. Typical values
of the stiffness of various types of packing are given in Table I.

**Table I.—Stiffness Values (approximate) of Typical Head Packings.**
(lb. per square inch per inch)

<table>
<thead>
<tr>
<th>Head packing (or cushion)</th>
<th>lb. per square inch after 25 blows</th>
<th>after 2000 blows</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-in. felt</td>
<td>7700</td>
<td>17,200</td>
</tr>
<tr>
<td>3-in. coiled hemp rope</td>
<td>8300</td>
<td>12,100</td>
</tr>
<tr>
<td>2-in. sacking</td>
<td>9600</td>
<td>15,000</td>
</tr>
<tr>
<td>3-in. asbestos fibre</td>
<td>7900</td>
<td>8,300</td>
</tr>
<tr>
<td>4-in. softwood</td>
<td>3000*</td>
<td>32,000*</td>
</tr>
</tbody>
</table>

**Notes.—** The thickness is measured under the static weight
of the hammer after 25 blows.

* Subject to wide variation.

It is necessary to make allowance for the effect of the helmet on the impact
when determining the effective free fall $h_1$ of the hammer as given by

$$h_1 = \left( \frac{M}{M + m} \right)^2 h.$$

In addition to the effect of the helmet and the ratio of the weight of the
hammer and helmet, the effective free fall of a drop-hammer is about 80 per
cent. of the actual fall and that of a steam-hammer about 90 per cent. Either
reduction factor can readily be taken into account, by reading from the appro-
piate scale, when using the charts in Fig. 8. The reduction factor for a diesel
hammer should be obtained from the makers.

It should be noted that with the stress-wave method, as with Hiley's formula,
the quake of the soil should be measured or an estimate of the amount must
be made. If the quake cannot be determined accurately, a reasonable dimension
for $C_s$ is 0·1 in. and it may be seen, by reference to data compiled by Bullen,
that this will possibly exceed the actual amount and the results may therefore be slightly conservative. It is important to note that the equivalent elastic set, as used by Dr. Glanville, is (twice the actual plastic set) plus (the quake or elastic set of the soil); therefore \( e = 2s + C_a \).

The application of the chart is shown in the example which follows.

**Example.**—Determine (a) the greatest allowable free fall of a 4\(\text{4} \) ton drop-hammer, (b) the least final actual (plastic) set, and (c) the resistance to penetration of a 16-in. square reinforced concrete pile 70 ft. long. The maximum allowable stress during driving is 2500 lb. per square inch, the weight of the helmet is 10 cwt., the packing on the head comprises 2 in. of compressed sacking, and assume that the quake of the ground has been determined to be 0\(\text{.15} \) in.

The weight of the pile is \( \frac{16 \times 16 \times 150}{144} = 267 \) lb. per foot.

The ratio \( \frac{\text{weight of helmet}}{\text{weight of hammer}} = \frac{m}{M} = \frac{0.5}{4.5} = 0.11. \)

The ratio \( \frac{\text{weight of hammer and helmet}}{\text{weight of pile per foot}} = \frac{M + m}{w} = \frac{5 \times 2240}{267} = 42. \)

Assume the stiffness of 2 in. of compressed sacking is 11,000 lb. per square inch per inch after about 500 blows (see Table I).

Referring to Fig. 8, from charts B and C, the greatest allowable free fall is 3 ft. 6 in. Also from charts A and E, the least plastic set for a quake of 0\(\text{.15} \) in. is 0\(\text{.19} \) in., that is approximately five blows per inch of penetration. The greatest force that may be applied at the top of the pile is \( \frac{2500 \times 16^2}{2240} = 286 \) tons, which represents a measure of the total resistance to penetration (including side friction if any). When allowance is made for the inertia force accelerating the pile, say 30 per cent., the resistance to penetration is given approximately by \( R = 0.7 \times 286 = 200 \) tons.

The charts in Fig. 8 may, of course, be used to determine any two of the four data, that is the weight of hammer, distance of free fall, the least stiffness of packing required, and the set, provided the other two terms are known.

Since the chart is based on driving to obtain a stress of 2500 lb. per square inch, it cannot be readily adapted to apply to piles driven with smaller stresses, but it is useful to safeguard against greater stresses occurring which may needlessly damage the pile.

**Weight of Hammer.**

It is always better to use a heavy hammer with a small fall or soft head-packing rather than the contrary; see also the comments on this matter in connection with sheet-piles (page 44). Code No. 4 (2.8) recommends that the weight of the hammer should be at least half that of the pile and that the efficiency of the blow (\( \eta \) in Hiley’s formula) should be not less than 30 per cent., or both restrictions should apply. (The writer feels that Code No. 4 should be disregarded where it refers to Hiley’s formula and driving stress.) Notwithstanding the difference between the knowledge of driving stresses obtained more recently and
the beliefs held previously, to adopt this ratio of weights always promotes efficient driving. The recommendation of having a hammer of sufficient weight to give a set of not less than 0.1 r in. (except when a pile is driven down to rock) is sound as it enables the determination of the resistance to penetration to be more accurate than if based on conditions at refusal when it is necessary for the factors $C_1$, $C_2$, and $C_3$ to be most accurate.

For precast concrete piles the Code recommends that the weight of the hammer should be not less than thirty times the weight of 1 ft. length of the pile and, if long followers are used, allowance should be made for the reduction of efficiency of the blow; the method to be adopted is not stated. The loss of efficiency is expected to be less than that obtained by adjusting the factor for the efficiency of the blow as given by Hiley and by adding the temporary compression of the follower to $C_p$ in Hiley's formula. Applying the charts in Fig. 8, the follower must be considered to be part of the helmet owing to the free joint between it and the pile, but the temporary compression must be allowed for, preferably by considering it to be additional to that of the head-packing.

**Tension in Piles during Driving.**

The stress-wave theory indicates the possibility of tension occurring in a pile during driving with a normal hammer-blow causing rapid penetration into soft soil. A wave of tension is reflected from the foot of the pile in this case instead of compression. Glanville had previously referred to the possible occurrence of longitudinal tension. Mr. E. A. Smith in a test without side friction obtained a tensile stress in the concrete of about 600 lb. per square inch. Normally, friction reduces the tensile stress. It is good practice to reduce the energy of the blows until there is enough resistance to permit a moderate set for each blow; with a steam-hammer, more head-packing may have to be provided or other means adopted to soften the blow. Prestressed concrete piles have an advantage in these conditions over ordinary reinforced concrete piles.

**Driving Piles in Tough Clay.**

General experience is that drop-hammers or single-acting steam-hammers are better for driving into clay and a relatively heavy hammer is more necessary than in other types of soil. With most clay no consolidation of the soil is obtained during the driving and the surface is lifted in the vicinity of the piles by an amount equal to the volume of the piles driven into the clay. Cases are known where the clay is forced laterally and has been known to displace horizontally a retaining wall or sheet-piling or foundations of a structure at some distance from where the piles are being driven. Driving sheet-piles into tough clay does not generally require precautions to prevent these troubles, but with bearing piles great resistance to driving is sometimes met. As an example, when driving 14-in. H-section steel piles and 14-in. diameter cylindrical fluted tubes piles for the Duluth–Superior bridge difficulties were experienced. Not much trouble was experienced in driving the H-piles with a diesel hammer having a rated energy of 30,000 ft.-lb., but considerable difficulty was experienced in driving the tube piles which were of metal ½ in. thick and tapered from 14 in. diameter down to 8 in. A single-acting steam-hammer of 24,000 ft.-lb. of energy did not produce
appreciable penetration, although it was able to drive the H-piles. The method eventually used and which is reported as being successful, was to use a 10-in. diameter auger 52 ft. long, by means of which a 10-in. diameter hole could be drilled through 40 ft. to 50 ft. of clay in seven to ten minutes.

A method of driving piles by vibration is described on page 157.

**Short Dollies.**

Short dollies, which are necessary to provide a cushion which saves the helmet from being damaged by the hammer, may be of hardwood (Fig. 9b), such as pynkado which is tough, or of elm which is not quite so resistant but deteriorates rapidly during hard driving; the cost of, and the loss of time during, replacement may be onerous. A recent development is to provide laminations of fabric impregnated with synthetic resin complying with B.S. No. 668 or No. 972 (type C) and thin sheets of hardwood, as shown in Fig. 9a; micarta is used similarly in the U.S.A. This type of dolly has been found to withstand the driving of over a thousand piles and to be well worth the extra initial cost.

**Bored Piles.**

There are a number of proprietary types of bored piles, varying in diameter from 8 in. to upwards of 5 ft., some of the more common types of which are described in other publications. For short piles in clay and for small loads boring by mechanical augers is common. Such piles, which are an alternative to deep footings or piers, have been found by the writer to be quick and economical for buildings of light weight on soil which is cohesive enough that it does not immediately fall into the open bored hole. Mobile drilling rigs, either self-propelled on tracks, as in Fig. 10, or mounted on trucks, are also used for boring for piles up to 24 in. diameter. Short piles can be bored by machine-operated augers. The Building Research Station estimate that two men using hand equipment take 40 to 80 minutes to bore a pile in London brown clay depending on the size, say, 10 in. to 14 in. diameter, and 8 ft. to 14 ft. long.
The bearing value of bored piles largely in clay is usually low as it depends only on the cohesive strength. A static formula giving the ultimate load \( R \) when \( c_1 \) is the cohesive strength of the clay at the foot of the pile, \( c_2 \) the average strength in the depth of the pile, \( A_p \) the cross-sectional area, and \( A_s \) the surface area of the pile, is

\[
R = 9c_1A_p + 0.4c_2A_s.
\]

Factors of safety of 3 on the end bearing and 2 or 2\( \frac{1}{2} \) on the side friction are typical. The adhesion of the soil within 5 ft. of the surface is usually ignored. It is important that the concrete is rammed in position to ensure no air locks to cause voids.

**Fig. 10.**

**Large Piles or Piers.**

For larger and deeper bored piles, to carry loads up to 2000 tons, the soil is removed by means of either a percussion grab dropped down inside a tube, or by a high-speed auger.

Boring is carried out by means of a cutting edge attached to a lining tube which is driven into the ground with a combined downward and oscillating motion. The hammer-grab operates within the tube and, if necessary, is fitted with special rock-breaking points and chisels. The piles may be 27 in., 36 in., or 39 in. (1 metre) in diameter. The pile is generally formed by placing the concrete with a tremie. The skin friction of the concrete pile is greatly increased by the compaction and ridging which is produced by the end of the tube during the withdrawal operation. A similar plant developed in Great Britain is shown in Fig. 11. Rapid sinking and concreting is possible in this way. Alternatively, the aggregates may be placed first and colloidal grout injected through tubes from the bottom of the pile upwards. In the case of a 52-ft. test pile at Rogerstone Power Station, a pile of the French type of 1 m. diameter in soils varying from loam to shingle and marl developed a skin friction of 500 tons, bearing on the bottom of the pile having been largely prevented, during test, by means of a plug of vermiculite concrete. Such piles are suitable for carrying large loads down to a stratum of high bearing capacity such as rock.

Another type of pile applicable to those conditions is the Gow caisson, which is used in America and consists of steel sections 16 ft. long; the diameter varies from 58 in. to 82 in. at the top, decreasing 2 in. for each section downwards.
They are sunk by drilling with a 1\(\frac{1}{2}\)-cu. yd. bottom-dump auger mounted at the bottom of a telescopic shaft which, in addition to being lifted and lowered, can also rotate. During clockwise rotations, the blade cuts into the ground and the material drops into the auger. After the top part has been bored to a depth of about 16 ft., and the first 16-ft. section of lining is in place, the next part is bored, but 2 in. smaller in diameter, and the second 16-ft. section is placed. If an enlarged base is required, undercutting is carried out by hand at the bottom of the bore. The concrete is placed down a chute, the end of which is kept about 4 ft. above the top of the concrete, and the linings are retracted by the use of extracting-gear on shear-legs having a maximum uplift of about 100 tons.

Cylindrical foundation piers of depths up to 80 ft. have been constructed for buildings in London and elsewhere by means of the plant shown in Figs. 12 and 13.\(^{(2,13)}\) The auger (Fig. 12) is from 3 ft. to 7 ft. in diameter and is attached to the end of a vertical hollow square steel shaft suspended from the crane jib.
of an excavating machine. A cantilevered member carrying the guide and rotating mechanism for the shaft is fixed to the excavator, the motor or engine and winches of which provide the power and motions for the shaft and auger. The auger is worked into the ground by screw-action aided by the weight of the shaft. In very hard ground the sinking can be assisted by an hydraulic jacking device attached to the guide. The length of the shaft, within which is another shaft, is about 50 ft. When the hole has been bored to this depth the inner shaft comes into operation automatically and boring proceeds to the depth required. The helical blade of the auger comprises about two complete turns. The auger rotates clockwise when boring and after each downward travel equal to about the length of the auger the rotation is stopped and the auger is brought to the surface (Fig. 12), the jib is swung round and, by an anti-clockwise rotation of the shaft and auger, the excavated soil on the screw is flung off. The jib is then swung back and the auger is lowered into the bore to make a further cut. If the area at the base of the pier is required to be greater than that of the hole excavated by the auger, a device (Fig. 13) comprising cutters projecting from a steel cylinder is lowered to the bottom of the bore and rotated. The link-motion to which the cutters are attached causes them to extend progressively

![Image](image_url)

Fig. 15.

farther out of the cylinder as the side of the bore is excavated so that an enlarged cavity is formed in the ground. The diameter of the largest cavity which can be formed is about 15 ft.

The bore, and the cavity (if any) at the base, are filled either by dropping concrete through a sheet-metal funnel at the top of the bore or by lowering it in bottom-opening skips.

An American machine (Figs. 14 and 15) for boring similar holes is also used in Great Britain (2,14) and comprises a chassis with hydraulic stabilising jacks, surmounted by a retractable jib 78 ft. long which is in a horizontal position during transit (Fig. 15). At the position of each pier the machine is levelled by the jacks and the jib is raised to a vertical position. Within the jib is a square steel kelly-bar which is rotated by means of a rotatable table operated by a diesel engine. A hard-steel auger of the diameter required is attached to the bottom of the kelly-bar by a quick-release pin, and when it is rotated it bores into the ground at a speed depending on the hardness of the material. The auger is raised to the surface at short intervals, and by a rapid rotational movement the excavated material is flung off the blades and falls around the perimeter of the hole (Fig. 14). When the hole has been drilled to the depth required the auger is replaced by an under-cutting tool, which flares out the bottom of the hole to
a greater diameter, thus providing an increased bearing area for the pier. Upon completion of the boring operation reinforcement, if required, is lowered into the hole and the entire excavation is filled with concrete. The machine can drill holes up to 8 ft. diameter.

The equipment shown in Fig. 16 comprises a crane mounted on an endless track and, suspended from the crane, a drill called a "turn-grab". Cylindrical steel casings are forced into the ground by means of hydraulic jacks and the soil within the casing is excavated by the grab progressively as successive lengths of casing are sunk. The grab is attached to a square kelly-bar and has three blades which are opened and rotated when excavating (to the full diameter of the casing); one revolution, or less, is sufficient to fill the grab, which can operate in any type of ground from silt to soft rock. The casing is then filled with concrete deposited from a self-opening skip and, except in ground which is not firm, the casing is extracted. The diameter of the casing is generally 4 ft. 2 in., and piles up to 100 ft. in length and supporting from 350 tons to 1200 tons each can be formed.

Safe Load on Large Piles.—The safe load on an augered pile of large diameter in clay is determined by the greatest amount of settlement acceptable.
The probable consolidation under pressure of the deeper strata in course of
time must be allowed for and the consolidation will tend to be slow in the case
of bearing in a thick stratum of clay as is the case of London blue clay. Con-
solidation could be rapid in a similar bed of clay with seams of sand. If the
end bearing, whether the pile is under-reamed to a larger diameter or not, is
assumed to have a unit ultimate bearing capacity of \(9\sigma_c\) where \(\sigma_c\) is the cohesive
strength, a factor of safety of three gives a safe load of \(3\sigma_c\), which may represent
an intensity of a safe load of 7 tons per square foot, if the clay increases in stiffness
with increase of depth. Whether friction on the side can be added to this load
if the base is enlarged is a matter of controversy at present. Tests cannot prove
whether the end bearing will continue indefinitely to act at its initial value or
will tend to decrease as consolidation occurs so that the ultimate side friction
is developed. However, for London blue clay the maximum skin friction may
be between \(0.3\sigma_c\) and \(0.6\sigma_c\) with a maximum of 2000 lb. per square foot.

If augered piles or similar piles of large diameter are entirely in clay and
do not bear on a hard stratum, for example rock or a thick bed of compacted
gravel, they will have much less frictional support compared with the several
bored or driven piles they may appear to replace. Most of their resistance to
penetration is due to end bearing which in clay does not often increase much
with increase in depth. Such large piles then produce bulbs of pressure at the
foot of the piles similar or more intense than if the piles were omitted, and there
is the possibility of greater settlement than with ordinary piles. Accordingly
all types of large piles are most suitable for sites where there is an end bearing
on rock, or compacted granular soil of sufficient thickness.

Cooling Large Piles.—Owing to the heat generated by the setting of the
cement in concrete piles of over 3 ft. in. diameter, it is a desirable precaution,
which involves no practical loss of strength, to form a core of 6 in. diameter,
or slightly more, into which water can be circulated from a \(1\frac{1}{2}\)-in. delivery pipe
extending nearly to the bottom; a steady flow should be maintained for about
the first three days after placing of the concrete. It has been questioned whether
the heat generated by the setting of the cement in a 4-ft. diameter pile, or greater
could cause a reduction of the friction between the clay and the concrete, but
since the water-cement ratio required for hydration of the cement is only about
0.2 and the water-cement ratio of the concrete when placed may be about 0.6,
there may be no serious difference between the content of free water in the concrete
after hardening and the moisture content of the clay.

Buckling of Piles.

Buckling of piles under axial load in a soft soil is a rare occurrence because
even soft clay and mud provide some lateral restraint. Reference can be made
to the work of Cummings (2.15) and Granholm. However, for piles driven to
bear on rock and mostly in water, the “long-column” factors recommended in
B.S. Code No. 114 can be applied to concrete piles, or the Perry–Robertson
formula in B.S. No. 449 to steel piles. Bergfelt (2.16) has given reports of tests
of solid steel piles in soft clay and has related the buckling load to the shearing
strength of the surrounding soft clay.

Long piles unsupported laterally are dealt with on page 35.
Prestressed Concrete Piles.

In recent years there has been a great increase in the use of prestressed concrete piles. The advantage expected at first from their use, compared with precast reinforced concrete piles, was greater flexural strength which enables more slender piles to be safely transported and pitched. It was not certain what disadvantages might be encountered, for example, greater liability to the breaking of the heads of the piles during driving, owing to the initial prestress adding to the compressive stress induced by the driving. However, subject to extra binding at the head and foot of the pile, as in ordinary precast concrete piles, prestressed piles have been found to be at least equally resistant to hard driving as ordinary reinforced concrete piles. Prestressed piles are also much less liable to transverse cracking which may result from the wave of axial tensile stress reflected from the foot of the pile when driving against negligible resistance or from transverse vibration when the pile has penetrated very little and has no lateral restraint. For bearing piles the prestress is always uniform.

Earlier wide variations in the magnitude of the prestress are recognised as having been due to insufficient appreciation of the requirements. It is possible, though unusual, for a tensile stress of 600 lb. per square inch to be reflected from the foot of a pile. This can be avoided usually by reducing the energy of the blows of the hammer while the pile is meeting little resistance. A prestress of this magnitude, after losses, is desirable to prevent or reduce cracking caused by transverse vibration of the pile, and is about the minimum required to avoid cracks while being transported and pitched. British experience of this was confirmed before any corresponding American experience had been obtained. In American practice there were many cases in which during driving horizontal cracks occurred with a prestress of 400 lb. per square inch or less, and it has been confirmed that 700 lb. per square inch can be considered as a safe lower limit of stress for normal driving. British experience shows that generally 1200 lb. per square inch is the desirable upper limit of initial stress and 800 lb. per square inch after all losses have occurred.

To avoid breaking the heads of piles during driving, closely-spaced binding is desirable for not less than 2 ft. from the head. Theoretically the instantaneous compressive stress at the foot of the pile can be double that at the head, if no side friction is developed and the pile is driven to refusal. In practice, this is only likely in very unusual cases, which the engineer will usually be able to anticipate, such as meeting rock after a very short penetration. However, it is more often possible that stresses at the foot may equal the stresses at the head and, unless there is substantial penetration in soil giving significant side friction, binding at least equal to that usual with ordinary concrete piles should be provided at the foot of a prestressed pile.

The steel in a prestressed concrete pile can be either pretensioned or post-tensioned, and in the latter case the pile can be in several lengths stressed together before pitching. If prestressed by bars, the pile can be extended after driving by attaching another length. The ability to joint with plastic and extend a pile by 30 ft. or 40 ft. and commence redriving less than an hour later enables for example, 100-ft. piles to be driven with a 60-ft. piling-frame. To have the flexural strength to prevent cracking during further driving, the extra length
must be stressed on to the length or lengths already driven, and this is most conveniently done by providing a small platform and lifting tackle for the jack near the top of the piling-frame.

In Fig. 17 the cross-sections of typical solid piles with pre-tensioned and post-tensioned steel are shown, together with examples of hollow and tubular piles with pre-tensioned and post-tensioned bars suitable for long piles and in cases where greater strength in bending during pitching is necessary or where greater surface area is required to provide increased frictional resistance. With

pre-tensioning it is possible to extend a pile by a thin steel sleeve and fine concrete at the contact face \(^{(2-17)}\). Where penetration into rock is desired it is possible to embed in the end of a pile with pre-tensioned steel a length of H-section steel joist as has been done successfully near San Francisco. Piles with post-tensioned steel and of cylindrical cross-section can be made from a number of short cylindrical concrete sections, post-tensioned together and the ducts grouted before transporting to the site and pitching and driving; this method was adopted for the Lake Ponchartrain Bridge. The post-tensioned steel can be either wires, strands or high-tensile bars, the last being similar to the methods used for the cylinders, described on page 198, for a jetty at Erith, and, with a different method of handling, at Colombo Harbour \(^{(2-18)}\).

Great importance must be attached to the method of making the joints to obtain even bearing and to prevent the joint being the cause of future corrosion of the prestressing steel. If some steel should corrode, the wires or other tendons will quickly fail, owing to the relatively high working stress of about 0.7 of the tensile strength (before losses). Failure of some tendons on one part of the

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**Fig. 17.**

A.—Pile with pre-tensioned wires or strand.
B.—Hollow octagonal pile with post-tensioned wires or high-tensile bars, or pre-tensioned strand.
C.—Pile with central post-tensioned high-tensile bar (suitable for adding extra length).
D.—Hollow cylindrical pile with post-tensioned high-tensile bars or wires.
BEARING PILES

circumference would immediately cause an eccentric prestress which is undesirable. Although high-tensile bars are much less affected by corrosion, similar precautions are desirable. Generally tubular piles are driven with closed ends and cylindrical piles with open ends, in fact, common soil conditions do not permit any alternative. When driving a cylindrical pile in water, it may be desirable to have short connecting holes through the wall of the pile at intervals to maintain the water level the same inside and outside, thereby preventing damage from “water-hammer”.

### TABLE II.—PROPERTIES OF TYPICAL PRESTRESSED BEARING PILES.

<table>
<thead>
<tr>
<th>Shape of Pile</th>
<th>Cross-section of Pile (in.)</th>
<th>Maximum Length for One-point Slinging (ft.)</th>
<th>Pre-tensioned</th>
<th>Post-tensioned</th>
<th>Approx. maximum moment of resistance (in-lb.)</th>
<th>Approx. Safe Load when wholly embedded in Soil tons</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>No. of 0.2-in. Wires</td>
<td>No. of 0.276-in. Wires</td>
<td>No. and Size (in.) of Macalloy bars</td>
<td></td>
</tr>
<tr>
<td>Square (solid)</td>
<td>10</td>
<td>47</td>
<td>16</td>
<td>10</td>
<td>1-1 1/4</td>
<td>150,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>55</td>
<td>24</td>
<td>14</td>
<td>1-1 1/4</td>
<td>202,000</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>49</td>
<td>20</td>
<td>12</td>
<td>1-1 1/4</td>
<td>240,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>58</td>
<td>32</td>
<td>18</td>
<td>2-1</td>
<td>330,000</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>60</td>
<td>32</td>
<td>18</td>
<td>2-1</td>
<td>420,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>72</td>
<td>52</td>
<td>28</td>
<td>2-1 1/4</td>
<td>600,000</td>
</tr>
<tr>
<td>Circular or Octagonal with 8 in.-dia. circular hole.</td>
<td>16</td>
<td>62</td>
<td>—</td>
<td>14</td>
<td>1-1 1/4</td>
<td>345,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>78</td>
<td>—</td>
<td>24</td>
<td>2-1 1/4</td>
<td>520,000</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>58</td>
<td>—</td>
<td>14</td>
<td>1-1 1/4</td>
<td>400,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>70</td>
<td>—</td>
<td>24</td>
<td>2-1 1/4</td>
<td>575,000</td>
</tr>
<tr>
<td>Circular with internal diameter 10 in. less than external diameter.</td>
<td>36</td>
<td>100</td>
<td>—</td>
<td>36</td>
<td>4-1</td>
<td>2,600,000</td>
</tr>
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<td></td>
<td></td>
<td>115</td>
<td>—</td>
<td>54</td>
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<td>60</td>
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<td>—</td>
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<td>6-1</td>
<td>7,500,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150</td>
<td>—</td>
<td>94</td>
<td>8-1 1/4</td>
<td>11,000,000</td>
</tr>
</tbody>
</table>

Table II gives properties of typical prestressed concrete piles of various types with the alternative prestressing tendons of wire or bars where applicable. The table is based on conservative assumptions, and slight economy could be obtained in cases where it is possible to avoid tension during driving by adopting a little smaller stress than the lower prestress of the alternative designs given in the table.

For the piles with pre-tensioned steel, the initial tension in high-tensile wire of 0.2 in. diameter has been taken as 70 tons per square inch with 25 per cent. for losses due to creep and shrinkage. For the bending stresses during pitching, one-point lifting at 0.3 of the length from the head has been assumed with a limiting tensile stress of 325 lb. per square inch. For a permanent bending moment the tensile stresses will be greater. The approximate axial
safe load on the pile, disregarding the resistance of the soil which may be less, has been taken as 2000 lb. per square inch, minus the residual prestress. For the piles with post-tensioned Macalloy high-tensile bars, the initial tension has been taken as 45 tons per square inch. The moments of resistance stated allow for a tensile stress of 225 lb. per square inch not being exceeded, and the conditions for pitching and for maximum axial load are the same as given above for piles with pre-tensioned steel. For piles with post-tensioned wire, an initial stress of 64 tons per square inch has been taken for wire of 0.276 in. diameter minus 15 per cent. for creep and shrinking. For two-point support of the piles during pitching, the maximum lengths given in the table are increased by about 40 per cent.

BIBLIOGRAPHICAL REFERENCES (CHAPTER II).

2.2—"The Resistance of Piles to Penetration." By Russell V. Allin.
2.8—"Foundations." Civil Eng. Code of Practice No. 4. 1954.
2.11—"Concrete Year-Book" (Annually).
2.12—"The Short Bored Pile Foundation." Building Research Station Digest No. 42. 1957. See also publication (1961) by the Reinf. Conc. Ass. relating to Symposium on Large Bored Piles.
CHAPTER III

SHEET-PILING

Sheet-piling comprises a row of piles engaging with or interlocked with one another so as to form a continuous wall, which may be a permanent retaining wall, a cofferdam, or a river wall. The applications of sheet-piling to other uses, such as for lining trenches, are only variations of the same design problem that arises with river walls.

Sheet-piles may be of timber, reinforced concrete, or steel. The choice will depend not only on the relative cost of the materials, but on the suitability of a particular material for the intended use, its durability, and, in the case of temporary work, the cost of withdrawal and the salvage value.

Reinforced concrete has displaced timber to a large extent because of its greater durability, particularly where timber would be subject to attack by marine borers. Reinforced concrete is also cheaper in first cost in areas where suitable timber is not readily available.

Steel sheet-piling is usually somewhat more expensive than reinforced concrete for permanent construction, but can be driven through highly-resistant strata, and it is in general use for temporary work because it can be fairly readily extracted and re-used a number of times, and has at the finish a salvage value. Where watertightness is necessary, as in the case of cofferdams, steel sheeting is in general use.

The advantages of sheet-piling over other types of walls are speed of construction, economy of material, and the omission of excavation and foundations.

Timber Piles.

In recent years timber sheet-piling has been used much less in permanent construction than formerly, reinforced concrete having taken its place owing to its greater durability, but for temporary works timber sheet-piles are still used because of their lightness and the consequent lightness of the pile-driving equipment required.

Two types of timber sheet-piles are shown at (a) and (b) in Fig. 18. Type (b), known as Wakefield sheet-piling, has been used for a long time in the United States. It is both stronger and cheaper than type (a) as well as having less tendency to twist or warp. Timber sheet-piles of plain rectangular section are also used for cofferdams of small height when a puddle clay filling is contained between two lines of sheet-piles.

Timber sheet piles with square ends may often be driven in soft soils without damage, but if the piles are large or long, or if the soil necessitates hard driving, the end must be protected by a covering of sheet-metal $\frac{1}{8}$ in. or $\frac{1}{4}$ in. thick forming a cutting edge as in Fig. 19 (c). The sheet-metal also prevents brooming of the timber. If the pile penetrates compact gravel or a stratum of shale, a cast-iron shoe as in Fig. 19 (a) may be necessary. A shoe with one sloping face only is usual for piles lining a trench excavation.
Fig. 18.—Typical Sections of Timber and Concrete Sheet-piles.

Fig. 19.—Types of Shoes for Sheet-piles.
For timber piles, and other uses of timber where alternate wetting and drying occur, the resistance to decay is greatest with teak, Australian hardwoods like ironbark, fallowwood, and then in descending order from jarrah and karri to Oregon pine and other softwoods. All softwoods need pressure or other effective method of creosoting to increase their durability.

For permanent construction, only a few varieties of timber are suitable in water where timber is subject to attack by marine borers. Generally, marine borers may be expected where the water is salt and reasonably clear, and timber will not be so liable to attack in polluted waterways. As attack by marine borers varies from time to time in given localities, and only comparatively expensive timbers such as greenheart and teak are generally immune, the use of timber sheet-piling has become of recent years mostly restricted to temporary works and to work on rivers and canals where the salinity of the water is too low for marine borers. Oregon pine or other softwood should be pressure creosoted if used for other than very temporary work. Timber piles, if effectively creosoted, have been found to have unexpected durability in harbours, even when exposed to marine borers, probably due partly to the contamination of the sea-water. As an example, piles, which had been in service for 68 years, and were exposed during the dismantling of Pier No. 3 in New York Harbour, were stated to be in remarkable condition and good for further use, although exposed to wear in the range of the tide.

Reinforced Concrete Sheet-piles.

Typical sections of reinforced concrete sheet-piles are shown in Fig. 18. That shown at (e) has been fairly extensively used, the circular gap formed between two successive piles being cleared after driving, say with a water jet, and then grouted up to unify the construction. The alternative section (d) is, however, preferred as it enables better alignment of the sheeting to be obtained especially if driving into coarse shingle.

Table III gives the properties of a selection of sheet-piles, and typical details of the reinforcement are shown in Fig. 20. The design shown in Fig. 20 (a) was that used for Dunston power station,\(^{(3.1)}\) while that shown in Fig. 20 (b) is a design prepared by the writer.

It is both usual and the best practice to reduce the heads of reinforced concrete sheet-piles as shown in Fig. 20 (a) to take a driving helmet, unless the driving is easy and a head-packing only is used. The section can in that case be uniform throughout to the head. The objections are, however, that the piles are then not so easy to guide and there is more to cut away to expose the main bars for concreting into the coping or capping beam. If the head is reduced in section and the driving is very hard, the shoulders should be sloped.

With concrete sheet-piles to be driven in soft soil a metal shoe is unnecessary. If the piles have to be driven to a set, say in compact gravel, or driven through any hard soil a cast-iron shoe, as shown in Fig. 19 (b), is used. When driving, the sloping edge of the shoe is on the far side from the pile already driven. The shoe of the pile first driven, however, is symmetrical, and is generally slightly sloped on both edges.

* References thus \(^{(3.1)}\) refer to Bibliography on page 42.
### Table III.—Properties of Reinforced Concrete Sheet-Piles.

<table>
<thead>
<tr>
<th>Width (in.)</th>
<th>2-(\frac{1}{8})&quot;</th>
<th>2-(\frac{3}{4})&quot;</th>
<th>2-(\frac{7}{8})&quot;</th>
<th>2-(\frac{1}{4})&quot;</th>
<th>2-(\frac{3}{4})&quot;</th>
<th>2-(\frac{7}{8})&quot;</th>
<th>3-(\frac{3}{4})&quot;</th>
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</thead>
<tbody>
<tr>
<td>Thickness (in.)</td>
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<td>9</td>
<td>14</td>
<td>16</td>
<td>16</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td>Resisting moment per foot width in pounds-feet</td>
<td>5</td>
<td>5</td>
<td>6</td>
<td>6</td>
<td>7</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>1,900</td>
<td>2,530</td>
<td>3,200</td>
<td>3,950</td>
<td>4,800</td>
<td>5,480</td>
<td>6,040</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Width (in.)</th>
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<th>3-(\frac{3}{4})&quot;</th>
<th>3-(\frac{7}{8})&quot;</th>
<th>3-(\frac{1}{4})&quot;</th>
<th>3-(\frac{3}{4})&quot;</th>
<th>3-(\frac{7}{8})&quot;</th>
<th>3-(\frac{3}{4})&quot;</th>
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</thead>
<tbody>
<tr>
<td>Thickness (in.)</td>
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<td>20</td>
<td>20</td>
<td>20</td>
<td>22</td>
<td>20</td>
<td>20</td>
<td>16</td>
</tr>
<tr>
<td>Resisting moment per foot width in pounds-feet</td>
<td>9</td>
<td>9</td>
<td>10</td>
<td>10</td>
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<td>11</td>
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<td>12</td>
</tr>
<tr>
<td></td>
<td>8,700</td>
<td>10,500</td>
<td>11,900</td>
<td>15,500</td>
<td>19,800</td>
<td>21,800</td>
<td>24,300</td>
<td>30,300</td>
</tr>
</tbody>
</table>

Stresses: Concrete 750 lb. per square inch; Steel 18,000 lb. per square inch. (These stresses are conservative but are applicable to ordinary conditions.) Axial load not taken into account. Cover of concrete over bars: 1 in. or the diameter of the bar if greater than 1 in. Modular ratio: \( m = 15 \).

### Materials and Proportions of Concrete.

The materials and proportions of concrete suitable for sheet-piles are the same as for bearing piles. Opinions differ on this subject, but practice generally favours proportions of \( 1:1\frac{1}{4}:3 \), using rapid-hardening Portland cement if the piles are to be driven as soon as they have sufficiently matured. It is quite possible and reasonable to use proportions of \( 1:2:4 \) and ordinary Portland cement, generally with a richer mixture near the head, but the time of maturing is increased and thus delays the stripping of the pile shuttering and the handling of the piles. If time permits and greater durability is required, a mixture of \( 1:1\frac{1}{4}:3 \) is used with ordinary Portland cement. The shrinkage stresses and brittleness of the concrete are reduced by this means. For piles to be driven into sulphate-bearing soil, a sulphate-resistant cement should be used, although the cost may be a little greater.

For the detail design of reinforced concrete sheet-piles all the considerations affecting the design of reinforced concrete compression members apply, except that with hard driving the piles may be subject to stresses approaching the crushing strength of the concrete. The stresses during driving are dealt with elsewhere.(8,2) The area of the main reinforcement bars in slender piles should be not less than 2 per cent. of the cross-sectional area of the pile, or \( 1\frac{1}{4} \) per cent. if the slenderness ratio is between 30 and 40, and not less than \( 1\frac{1}{4} \) per cent. for stiffer piles.

For concrete bearing piles and normal driving conditions, the volume of lateral binding around the main longitudinal reinforcement, ignoring the short hooked ends of the binding, should be about 0.17 to 0.22 per cent. of the volume of the concrete. More binding should be provided at the top and bottom of the pile as shown in Fig. 20. For concrete sheet-piles it is, however, sometimes
Fig. 20.—Typical Details of Reinforced Concrete Sheet-piles.
desirable to increase the amount of binding compared with bearing piles, since in the rectangular cross-section of sheet-piles the binding is generally less effective. The main bars may be tied together by diagonal wire ties at intervals as shown, but, to prevent them coming inwards during concreting, pressed-steel forks may be used in pairs holding apart diagonally opposite bars. The only objection to the pressed-steel spacers is that when cross-cracking of piles takes place, either by heavy or eccentric driving, the cracks generally occur at the points where the diagonal spacers are placed. Whatever method is used for fixing the main reinforcement bars it is important that the main bars are not permitted to be too close to the surface, because of risks of spalling of the concrete cover and of corrosion, or be inside of their correct position because of the considerable reduction in resisting moment compared with that calculated with the bars in their correct position.

All the bars in any one pile should be exactly the same length, and should be placed with one end bearing on the shoe. The cover of concrete over the ends of all bars at the top of the pile should be the same, say, 2 in. The amount of the cover of concrete at the sides of the main bars is a compromise between cost and effectiveness. The greater the cover the greater the amount of concrete, and therefore the greater the cost, unless the bars are placed further in from the face of the pile, in which case the moment of resistance is reduced. For sheet-piles in fresh water a cover of 1 in. may be sufficient with a concrete mixture of $1:1\frac{1}{2}:3$ and good workmanship and driving conditions which are not too severe. For piles in sea-water, a cover of 2 in. is generally provided but, if the concrete is fully compacted and has a low water-cement ratio, a cover of $1\frac{1}{2}$ in. may be just as effective.

![Figure 21. Reduction of Axial Load on Bearing Piles.](image)
Long Piles.

The load on a long slender pile, except where the pile is supported laterally by the ground or by bracing, must be less than the safe load as calculated for a short column. The reduction should be in accordance with curve T given in Fig. 21, in which curve S is based on British Standard Code of Practice No. 449 (1959) for mild steel and curve C is based on British Standard Code of Practice No. 114 (1957) for reinforced concrete. For the purpose of comparison, curve C has been modified by assuming that the radius of gyration is one-third of the least lateral dimension; the actual ratio varies from about 0.31 to 0.35 according to the amount and position of the reinforcement.

The effective length of a free-standing bearing pile may be assumed to be the length exposed plus, say, one-quarter of the length embedded in the ground. The part of a pile embedded in the ground can be excluded from the effective length, except for a short length immediately below the surface, provided no unbalanced lateral pressure is likely to be exerted on the piles by the soil.

Prestressed Concrete Sheet-piles.

The ability to take advantage of the high compressive strength of good concrete and the great durability of such concrete even when alternately wet and dry, makes prestressed concrete very suitable for sheet-piles. Generally, possibly thinner piles can be used so that a prestressed pile may be easier to handle, transport, and drive than an ordinary reinforced concrete pile. Prestressed piles

![Cross-sections of Prestressed Sheet-piles.](image)

are made either by the pre-tensioning or post-tensioning methods. For thin piles the former method is generally more suitable, because there is insufficient space for the end anchorages. Piles having pre-tensioned wires are usually more economical for piles less than 30 ft. long as is common for sheet-piles, particularly if existing prestressed beds are available. Piles with post-tensioned wires or bars are made economically at the site or in factories, particularly if long piles are required. Some typical cross-sections of prestressed sheet-piles are given in Fig. 22.

Concrete for prestressing and the permissible stresses are described in British Standard Code No. 115, but it is often common to require concrete having a crushing strength at twenty-eight days of 5000 lb. per square inch if determined by cylinders, or 6000 lb. per square inch if by cubes. The water-cement ratio
should be between 0.38 and 0.45, and the concrete should be compacted to its greatest density. In the case of concrete in contact with water, the cover of concrete over the wires should be not less than about 1 in.

The diameter of pre-tensioned wires is generally 0.12 in. to 0.2 in., larger wires not being generally suitable for sheet-piles as a high degree of bond is desirable to withstand the driving. Wire ropes (strands) are used extensively in America and are becoming more common in Great Britain. The average total initial prestress is preferably about 800 to 1000 lb. per square inch after allowing for losses for creep and shrinking which are greater with pre-tensioning than with post-tensioning. In some cases other stresses are used, but it must not be overlooked that the tensile stress induced in the concrete during handling, transporting, and pitching of prestressed piles often determines the lower limit of the prestress.

**Table IV.—Properties of Prestressed Concrete Sheet-piles.**

<table>
<thead>
<tr>
<th>Cross-section (in.)</th>
<th>No. of 0.20 in. dia. wires</th>
<th>Moment of resistance (ft.-lb. per foot width)</th>
<th>High-tensile bars</th>
<th>Moment of resistance (ft.-lb. per foot width)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 x 4</td>
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<td>12 x 6</td>
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<td></td>
</tr>
<tr>
<td>18 x 6</td>
<td>27</td>
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<td></td>
</tr>
<tr>
<td>18 x 9</td>
<td>40</td>
<td>19,100</td>
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</tr>
<tr>
<td>24 x 12</td>
<td>75</td>
<td>35,100</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Stresses.**

Compression in concrete (after losses): 2500 lb. per square inch.

Tension

- "", wires (70 tons per square inch at stressing): 52½ tons per square inch after 25 per cent. losses.

- "", bars (45 tons per square inch at stressing): 38½ tons per square inch after 15 per cent. losses.

The moment of resistance of some prestressed sheet piles at ordinary stresses are given in Table IV. The loss of prestress due to shrinking and creep of concrete are allowed for in all cases, and in the case of post-tensioned bars, the loss due to creep of the steel and elastic contraction of the concrete is taken into account.

Although it is not often necessary to lengthen sheet-piles, prestressed piles with post-tensioned steel can be easily lengthened and mild-steel reinforcement can be cast in the head to bond to the extension if required. Should the piles need to be shortened the end anchors are removed. If the grouting is effective and, as is usually the case, the driving has not been severe enough to destroy the bond, the strength in bending will not be impaired. In the case of post-tensioned bars, it is easy to remove and unstress the end anchor to avoid the tendency of the concrete to fly when cutting away the head, but with some other systems of post-tensioning this risk has to be taken. Piles with post-tensioned high-tensile bars can be extended by using couplers to attach an additional bar, and subsequently stressing them in the usual way, but this is not often needed.
owing to the small bending moments near the top of the pile. For the same reason piles with pre-tensioned steel can be shortened easily by cutting, and can be lengthened by adding concrete reinforced with mild steel.

In both types of pile, mild steel is generally only required for bonding into the coping or capping beam; it is only in exceptional cases that the question of extending sheet-piles may arise. It is desirable to provide links at the top and bottom of the pile as in a reinforced concrete pile.

Special Types of Concrete Piles.

Although free drainage through sheet-piling is generally most desirable, if sheet-piles are used to form a permanent cut-off, say under the apron of a dam, special means may be desirable to ensure watertightness. The following description refers to the piles shown in Fig. 23 and used for this purpose for the Assiut Barrage, Egypt.\(^{(3,4)}\)

![Diagram of Mandrel and Washing-out Grooves](image)

Fig. 23.—Reinforced Concrete Sheet-piles used for a Cut-off Wall in the Reconstruction of the Assiut Barrage, Egypt.

Bars were embedded in the piles to hold the steel mandrels. Recesses, cast in the end joints of the piles, took the mandrels, behind which were washing-out grooves. Special piles took up errors due to longitudinal creep and any errors in verticality. The steel mandrels extended the full length of the piles and were 4 in. by 4\(\frac{1}{2}\) in. in external section with a hollow core measuring 2\(\frac{3}{4}\) in. by 2\(\frac{1}{2}\) in. The plant used for sinking the piles consisted of a mechanical excavator, on crawler tracks, fitted with a 40-ft. jib for handling and pitching the piles and for operating the 2-ton drop-hammer. The jetting pipes were suspended from the jib. A pyramidal four-legged tubular frame, 25 ft. high, followed behind the navvy for the purpose of withdrawing the interlocking steel mandrels from between the driven piles, and also carried the grouting mixer, pump, pipes, and jets.

Before a pile was pitched, one of the steel mandrels, with a loose cast-iron shoe at the bottom end, was threaded on to the leading end and into the jaws of the stretcher bars in the pile. The pile was picked up and the tail end was threaded on to the mandrel in the leading end of the last driven pile. The pile was sunk under its own weight by means of water jets on both sides, with an occasional tap with the hammer if required. The sinking of the last 6 ft. 6 in. was carried out by means of the hammer only, the jetting pipes being held 6 ft. or so above the bottom of the pile.

Water-jet pipes were lowered into the grooves on both side of the mandrels, and washed out any sand or other materials above the cast-iron shoes at the bottom. The mandrel was then withdrawn 6 in. or 9 in., leaving behind the
cast-iron shoe which was held by the bottom stretcher bar. The shoe acted as a plug in the bottom. The water jets were continued until clear water issued from the inside and around the mandrels at the top. A 2-in. diameter pipe was then fitted to the top of the mandrel and connected to the grout pump. Cement grout was injected down the inside of the mandrel, the water jets in the grooves being gradually reduced until grout showed at the top, outside the mandrels. The jetting-pipes were then gradually withdrawn, and when thick grout appeared at the top, following the jetting-pipes, the mandrel was slowly withdrawn, grouting being continued until withdrawal was complete.

Fig. 24.—Reinforced Concrete Sheet-pile used for a Cut-off Wall near the River Mississippi.

Another example of a special type of reinforced concrete sheet-pile is shown in Fig. 24. This unusually heavy type of pile was used for the Cahokia power station, St. Louis (3,5), to protect the sub-soil from encroachment by the Mississippi river. These piles were 70 ft. to 75 ft. long, and were jetted through sand and gravel, the grooves being subsequently grouted to prevent seepage.

Steel Sheet-piles.

In Table V is shown a selection of steel sheet-pile sections with their properties. Most of these sections are frequently rolled but, as sections are sometimes listed although the rolls may not be available, enquiries should be made while new work is in the design stage.

Sections with an essentially flat surface are of use, particularly for trench linings where the transverse moments are small, giving for that type of work the most efficient use of the metal. Others with strong but free interlocks are specially suitable for cellular cofferdams referred to later. It will be noted that in some types the interlock occurs at the centroid of the combined sections.
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**LARSSSEN**

**FRODINGHAM**

**UNITED STATES STEEL**

Note.—The sections entitled “United States Steel” are typical of American piles, and in many cases the sections rolled by the Bethlehem Steel Co., and the Inland Steel Co., are identical. Sections similar to those of the Appleby-Frodingham Steel Co., Ltd., are rolled on the Continent, for example by the Belval Co., in Luxemburg, which Company together with the Lorraine-Escaut Co., in France, roll sections suitable for cellular cofferdams. A Senelle section produced by the latter company is illustrated in Fig. 108 (page 157).

In continental Europe similar sections of piles are in use and in fact several types were introduced into Great Britain from there. The Frodingham type is known in Germany as Hoesch; Larssen is known by the same name while the American sections, as MZ 38 and ZP 38, have equivalents in the Klöckner types.

The section modulus of sections marked MP is based on separate piles neglecting any interaction due to friction of the interlocks.
With the European sections it is frequently assumed that the full section modulus is developed, and the correctness of this assumption under normal conditions has been claimed to have been proved by practical experience in deep cofferdams and by tests. Tests in which the actual section modulus has been determined by careful measurement of the deflection of loaded piles are reported to have shown that even when the interlocks of the piles are quite free, and have been oiled to reduce friction, something like 60 per cent. of the combined section modulus is developed. A small amount of loose sand or other material, which helps to develop friction, may be expected to increase this figure to nearly 100 per cent. In a case in which, for example, the piling is driven through a stratum of mud and then rests on top of hard rock which it cannot penetrate, it is advisable to make a reduction of as much as 25 per cent. in the combined section modulus of the sheeting. Alternatively, adjacent piles can be welded to one another so as to ensure that no sliding takes place. It is also advisable to make a reduction when the piles cantilever and filling is placed on one side of the sheet-pile wall after driving. There is no reduction of efficiency if the interlocks are on both faces of the wall.

The moduli given in Table V are in each case from the maker's tables, but in the writer's opinion the American method of discounting completely friction in the interlock is too conservative, and the opposite applies to European sections which have central interlocks and for which the moduli are calculated on combined sections.

All makers of sheet-steel piling make special sections to form corners and T-connections. Standard details for splicing on additional lengths are available but, unless the connection has the same moment of inertia as the plain section, the ability to transmit the full moment is not obtained, so that it is always desirable to avoid splicing. If, however, splices must be provided, say, because

Fig. 25.—Methods of Forming Joints in Steel Sheet-piles.
the sheet-piles cannot be handled or pitched in one length, the joint should either be placed at a position of small bending moment [in which case a detail such as Fig. 25 (a) may be used] or the necessary strength obtained by increasing the number of bolts. Welded connections, although more expensive as the welds must be made at the site, are preferable to riveted or bolted connections for obtaining the full strength of the section for permanent work, but bolted joints are used where the top length is to be subsequently recovered.

In the case of some British types of sheet-piling the interlock, although rolled separately, is normally welded at the foot to the sheet-pile on one side, and the piles are driven in pairs. With these types of piles, when headroom is restricted, the upper part of the clutch of the leading edge of the piles being driven is omitted and this enables the next pair of piles to be engaged part way down. The upper part of the clutch is welded to the next pair of piles and, in pitching, is threaded over the exposed tongue of the leading edge of the pair of piles driven previously, so that when driving is completed the halves of the clutch abut. It will be noted that when this is done the piles are driven with the lower part of a clutch leading; normally a clutch is driven over the tongue of an interlock of a previously-driven pile.

There is normally no loss of strength in not jointing the clutch since the clutch acts merely as a locking device between adjacent sheet-piles.

**Corrosion of Steel Sheet-piles.**

The corrosion of steel sheet-piles occurs mostly within the tidal range and depends upon the properties of the water and the method of protecting the steel. Apart from the corrosive qualities of sea-water, which vary widely as has been shown by the reports of the Sea Action Committee of the Institution of Civil Engineers,\(^{(3,6)}\) the pollution of river waters and tidal waters, particularly by sewage and industrial waste, leads to widely varying degrees of attack. The rate of corrosion is about \(\frac{1}{50}\) in. per year in sea-water and \(\frac{1}{50}\) in. per year in fresh water, but these are not the limits. The initial protective treatment is important since subsequent maintenance is generally limited to the outer surface, and in tidal waters the interval between tides is too short to clean and repaint under satisfactory conditions.

The commencement of corrosion is due to electrolytic action and the presence of mill scale. The principal contributing agency, the mill scale, can be removed by exposing the pile for a long period. Usually time does not allow this, and the scale adheres too strongly to be removed except by pickling or sand-blasting. If the steel is sand-blasted, the corrosion-inhibiting properties of red lead is then usually sufficient to prevent from becoming active the particles of scale of pin-head size just below the surface. After thorough cleaning the steel should be primed with red-lead paint having a composition within the following limits: Genuine red-lead paste, 80 to 85 per cent.; pure linseed oil in the proportions of one part boiled to two parts raw, 20 to 15 per cent. Two coats should be given, especially if subsequent maintenance is impossible, and the steel should then be painted with one coat of coal-tar obtained from high-temperature distillation of coal in horizontal retorts and which has been treated with lime to neutralise acids. These conclusions have been supported by Gliddon and
Chabor (3.7) in Canada. For two-coat work the writer would prefer one of red lead and one of a good bituminous paint, but the second coat should be chosen for compatibility with red lead; for example, a paint containing naphtha might soften the red lead and should not be used. If steel has to be painted over a rusty surface a red-lead paint is practically the only priming paint worth the cost of applying. The assumption that corrosion occurs only at exposed surfaces does not apply if water and air have free access. The advantage of including 0.25 to 0.33 per cent. of copper in the steel to reduce corrosion has been described, but the results of tests (3.8) show that though the copper content is normally worth the small extra cost, it cannot be considered an alternative to proper painting (3.9).

If the appropriate corrosion rate previously mentioned is divided into half the thickness of the web of the steel pile it is intended to use, the probable life will be seen to be ample for many uses without any particular surface protection. Cathodic protection can be considered if the corrosive conditions are severe, but steel sheet-pile walls are not generally suitable for the location of the anodes (3.10).

**Under-water Cutting of Steel Piles.**

Under-water cutting of steel sheet-piling by means of oxy-acetylene flame is now usual; a submarine-type blow-pipe is used, an oxygen mask keeping the water from contact with the flame. This method is in common use for cutting steel sheeting in depths of water up to about 30 ft. For depths beyond this, an oxy-hydrogen flame can be used to the limit at which a diver can work with normal diving equipment. The thickness of steel sheet-piling to be cut under water will seldom exceed ½ in., but it is possible to cut steel 4 in. to 5 in. thick under water by this method.

**BIBLIOGRAPHICAL REFERENCES (Chapter III).**

CHAPTER IV

DRIVING SHEET-PILES

Effect of Pile-driving on Existing Structures.

The driving of piles may sometimes be objectionable either because of the noise and vibration or because of the direct effect of the driving on the sub-soil immediately surrounding the site. With regard to the former effects, there are advantages in using double-acting hammers for driving the sheeting. It is unusual for complaints to arise due solely to noise or vibration from this cause. The same cannot be said, however, of a drop-hammer which, while it may be unlikely to cause any serious damage, may shake loose objects in surrounding buildings and loosen plaster and other finishes insecurely bonded.

With regard to the direct effect of the driving of piles, in the case of steel sheeting there is seldom any trouble, but if piles of other materials are driven into clay the soil is generally displaced rather than consolidated or compressed. Either the surface of the soil immediately around the piles rises by as much in volume as the piles themselves displace, or there will be lateral movement of the soil which will tend to displace horizontally drains and other pipes or even structures near by.

Driving.

The piles should always be driven with the tongue leading. This rule applies to timber, concrete, or steel sheet-piles except, in the case of steel piles, as described on page 41. Sheet-piles of any material tend to drive outward and also to creep in the direction in which the wall is being driven. This action may be prevented by pitching and driving partly a few piles at the beginning of the wall and then completing the driving of these few in the reverse order, that is, from the last pile in the row back towards the original pile. If this is done, a clean groove or interlock may be obtained with steel piles by inserting a loose bolt or rivet at the bottom of the pile before commencing driving. If the piles lean in the direction of driving, the slope may be reduced and often corrected entirely by pulling on the heads of successive piles during driving, the pull being applied to the top of the pile being driven, or to the pile last driven, and in the direction of the first driven pile. The use of taper piles is a last resource. The interlock of steel piles ensures no creeping apart of successive piles, but with timber and concrete it is usual to taper the shoes, as shown in Fig. 19, to correct the tendency to creep sideways. Driving between temporary walings at the level of the pile-heads when driven, as shown in Fig. 26, is the preferable method of ensuring alignment. One or both temporary walings may subsequently be made permanent by bolting through the piles. If no other method of guiding is possible, as a makeshift, drifting out of line may be minimised by temporary steel clamps sliding against the last pile driven.

Drop-hammers are generally used for driving sheet-piles of any material in cohesive soils. Single-acting steam-hammers are more suitable for heavy piles and are therefore seldom used for steel sheet-piles.
Double-acting hammers are suitable for driving into granular soils because the rapidity of the blows cause vibrations in the ground which greatly assist penetration. The ratio of the weight of a drop-hammer to the weight of a steel or timber pile driven into cohesive soils, except in very soft ground, should be not less than $1\frac{1}{4}$ and preferably up to $2\frac{1}{4}$; for a concrete pile the ratio should be between $\frac{3}{8}$ and 1, and the drop should not exceed 3 ft. The ratio of the entire weight of a double-acting hammer (not only the weight of the moving parts) may be the same as for a drop-hammer if driving into granular soils, but in cohesive soils a greater ratio should be adopted. With single-acting hammers, the weight of the ram, which is generally about 40 per cent. of the total weight, should equal the weight of the pile.

Brief particulars of typical hammers are given in the following; dimensions and other details can be obtained from the literature issued by the makers.

Single-acting hammers of one type obtainable in Great Britain have weights of 4, 5\(\frac{1}{4}\), 8, 10 and 12 tons, the stroke being about 3 ft. and the number of blows delivered per minute being about sixty. For another British type the stroke is 4 ft. 6 in. and the number of blows is 36 for the lighter hammers and 24 for the heavier hammers; the weights range from 2 tons to 7 tons. Similar hammers in the U.S.A. have weights of 3, 4\(\frac{3}{4}\), 7 and 8 tons; the stroke of the lightest hammer is up to 2 ft. 5 in.; for the heaviest hammer, 3 ft. 3 in.; the number of blows is seventy for the lightest hammer and fifty for the heaviest.

The total weights of double-acting hammers obtainable in Great Britain range from 1\(\frac{1}{4}\) cwt. to 6\(\frac{1}{2}\) tons; the stroke ranges from 3 ft. 9 in. for the lightest hammer to 8 ft. 9 in. for a hammer weighing 1\(\frac{3}{4}\) tons, and to 19 ft. for the heaviest hammer; the corresponding number of blows are 500, 275, and 95 per minute, and the greatest energies per blow are 110, 2500, and 79,000 ft.-lb. respectively. Double-acting hammers obtainable in the U.S.A. range in total weight from 90 lb. to 9\(\frac{1}{2}\) tons; the stroke is 4 in. for the lightest hammer, 1 ft. for a hammer weighing 1\(\frac{3}{4}\) tons, and 3 ft. for the largest hammer; the corresponding numbers of blows are about 550, 200, and 85 per minute; the greatest energies per blow are 90, 2100, and 55,000 ft.-lb. respectively. Heavier double-acting hammers in the
U.S.A. range from 2 tons to 37 tons in weight; the strokes range from 10\(\frac{1}{2}\) in. to 16\(\frac{1}{2}\) in.; the numbers of blows per minute range from 150 to 100; and the energies per blow range from 3600 to 113,500 ft.-lb.

Diesel hammers are also used for driving steel sheet-piles. Since the number of blows per minute are generally less than for a steam or compressed-air double-acting hammer, a diesel hammer will not be so effective in soils, such as loose granular material, in which rapid vibration expedites driving. In other soils the weight of diesel hammer to be used may be determined by dividing the energy of the blow in foot-pounds by 9000, the result being the equivalent weight of a drop-hammer; more experience may show to what extent this rule can be applied by downward adjustment for steel sheet-piles in granular soils.

Continued blows on the helmet when an obstruction or sudden resistance to driving is encountered should be avoided to save possible damage to the foot of the piles and tearing open of the interlock, while any condition by which hammer bounce at the head of the pile is experienced should be avoided to prevent damage to the top of the pile. Hammer bounce shows that the pile has met either an obstruction or very stiff resistance. In the latter case it usually indicates that a heavier hammer is required and the change, by increasing the efficiency of the blow, will invariably give better penetration. Obstructions, such as old timber or boulders, encountered during driving, may sometimes be dislodged by ceasing driving the obstructed pile and driving the next.

Generally the driving of steel bearing piles and steel sheet-piles in a clayey subsoil does not develop so much frictional resistance that it is worth taking special means to reduce it. However, a method discovered in Indonesia and subsequently investigated at the University of Florida \(^{(4.7)}\) shows the possibility of making use of the principles of electro-osmosis (see page 126) provided the surface of the pile is an electrical conductor. In this application a relatively weak electric current is passed from an anode in the ground some feet away from the pile and the pile itself is made the negative pole. It has been stated the effect of the current in reducing friction is almost immediate, and if the current is reversed on completion of driving there is an immediate take-up on the pile. It was shown by the Indonesian tests that if metal strips were attached to concrete piles this method could be used successfully with them also.

**Jetting.**

Where hard driving is encountered, particularly in dense sand, water jetting may be necessary, as for bearing piles, especially as with sheet-piling a definite penetration usually has to be achieved. Jetting is seldom needed for steel sheet-piles but is more often used with concrete or heavy timber sheet-piles partly to avoid excessive driving stresses. The beneficial action of the water is generally considered to lie in the lubrication of the sides of the pile by the rising column of mixed soil and water, and for this to occur the soil must not be so porous that the water all escapes laterally, as it will do in hardcore or coarse shingle, but the volume must be sufficient to provide enough excess to come up around the pile; this is possible with most granular soils.

Jetting pipes may be cast in reinforced concrete piles as shown in section in Fig. 24, but this method is more expensive and not so efficient as a loose pipe.

\(^{*}\) References thus \(^{(4.1)}\) refer to Bibliography on page 56.
Normally it is only suitable if the soil or the site conditions would not permit of a loose pipe being used, since only a movable jet is of use to correct a pile running out of plumb. Apart from this exception for concrete piles, jetting for sheet-piles of any material is usually done by first opening a way into the soil with the jet and, after pitching the pile, working the jet down to the shoe of the pile, or beyond, both behind and in front of the pile. If the jet is used on one side only the pile will drive that way and it will be difficult to correct it. For this reason two jetting pipes are preferably used, and in any case the pipes are usually worked up and down while in use to keep a free path for the upward flow of soils and water around the pile. To enable this to be done, the jetting pipe or pipes are invariably

![Diagram](image)

**Fig. 27.—Method of Suspending Jetting Pipes.**

slung from the pile driver as in Fig. 27 whether the pump is on the frame or separate. The jetting pipes should be withdrawn before full penetration is reached to allow the ground to close in around the pile. The jetting pipe may be of 2-in. or 2⅛-in. bore extra strong pipe, with the end reduced to a nozzle outlet not exceeding one-quarter the area of the pipe.

The water consumption with a 1-in. nozzle may be about 140 or 150 gallons per minute at a pressure of 150 lb. per square inch. A duplex steam pump 6 in. by 4 in. by 6 in. (steam bore-water bore-stroke) is the minimum size that is considered useful for jetting, and 7½ in. by 4½ in. by 10 in. the minimum which should give the output and pressure mentioned. For a small jetting outfit, and if the boiler on the pile frame has ample spare steam-raising capacity, the duplex pump can be mounted on the pile frame between the winch and the leaders; normally, however, this method will only be satisfactory if the quantity of jetting water required is only small, otherwise there is a risk of delay due to shortage of the supply of steam. If the volume of water rising around the pile is insufficient
the jetting pipes must be kept moving to prevent them from "freezing". Because the steam supply needs to be considerable, for example, a 60-h.p. boiler in the latter case, other means of pumping should be used when circumstances permit. A two-stage centrifugal pump may be used and, where electric power is available, direct-connected pump sets of this type are generally much to be preferred because of the considerable reduction in weight and bulk of the equipment, principally by elimination of the boiler.

The two stages of the pump are sometimes built into one pump unit as in Fig. 28 and, as the centrifugal pump is suitable for speeds of about 3000 r.p.m., the unit is best driven directly by electric motor. Alternatively the two-stage pump and the electric motor are built in one compact unit (Fig. 29). Pumps of either type are made in a range of sizes but, for jetting, the larger sizes having 3-in. or 4-in. suction and 2¾-in. or 3-in. delivery are necessary and the power consumption will be 15 to 30 h.p. according to the duty.

Jetting greatly facilitates the actual driving, but as the piling gang requires several extra men the cost of driving is increased; on the other hand it is sometimes the only way to obtain the required penetration without excessive driving stresses in the piles.

**Submerged Driving.**

If it is necessary to drive below water level and a pile-frame is being used, a follower or long dolly, of such length that the top end remains in the leaders of the pile-frame when the pile is fully driven, can be used between the hammer and the pile-head. If extending leaders are used the bottom end of the follower and the top of the pile may be guided by the interposition of a helmet, but if not, and unless the driving is very easy, the pile will be most difficult to control in
direction. The follower may be of elm or other tough timber, or in the case of steel piles a length of similar steel pile section with cover plates to fit over the sheeting being driven, as shown in Fig. 30, thereby also fitting the driving helmet or the base of a double-acting hammer at the top end.

Driving by a double-acting hammer using compressed air when submerged is an alternative.

![Diagram of sheet pile being driven](image)

**Fig. 30.—"Follower" for Driving Steel Sheet-piles below the Leaders.**

Certain double-acting hammers operate successfully down to 70 ft. below water level when operating with compressed air, and 40 ft. with steam, after which trouble from condensation is usually met. A small air line is sometimes used to prevent water from entering the hammer cylinder via the glands.

**Driving to Curves.**

There is usually enough latitude in the interlocks of steel piles and enough play in the joints of concrete piles for a certain amount of curvature in plan to be obtained when desired. With timber piles, previous chamfering of the engaging parts is necessary, and with both concrete and steel special sections are necessary for sharp curvature.

For steel sheet-piles the diameter of circle that can be driven without using special piles depends on the play in the interlock. About 10 deg. swing is a typical limit for free interlocks, and this results in a minimum neat diameter inside the piling, for flat piling sections, for example, MP101 or MP102 in Table V, of 1.4 ft. 0.4 in. For types having less freedom in the interlocks, as IA, IB, 2 to 5 in Table V when in long lengths, the minimum diameter is about 60 ft., but
reduces for shorter piles, say, under 30 ft. long. For full circles an even number of piles is, of course, necessary for types having interlocks on alternate faces and special closure piles may be required.

Obviously the somewhat opposing properties of swing in the interlock, strength of the interlock or clutch, and a close fit for watertightness, involve a compromise. Strength of the interlock in tension normally only becomes of importance with sheet-piling driven to circles in plan and subject to the lateral pressure of the soil retained within cells so formed. This type of construction is not favoured in this country, but in America some types of sheet-piling are designed to have strength in tension through the interlock of, for example, about 8000 lb. per inch run in the case of types Mr01 and Mr02. For sections with a large modulus-to-weight ratio, the strength of the interlock is comparatively small, and in addition, due to the shape of the piling in section, there is a certain amount of resilience when subjected to transverse tension. If it is intended to stress such sections in transverse tension the makers should be consulted about the suitability of the particular section.

Fig. 31.—Double-acting Steam Hammer Driving Steel Sheet-piles.
Driving Steel Sheet-piles.

Steel sheeting may be driven either by a pile-frame with one of several types of hammers, or from a derrick from which is suspended an automatic double-acting hammer (Fig. 31). The interlock then serves as a lead for guiding the pile. The choice of driving plant for small contracts is frequently a question of the plant most readily available, but otherwise a drop-hammer is preferable for clay or marl and a double-acting hammer for non-cohesive soils.

Fig. 32.—Double-acting Steam Hammer used with Piling-frame.

Fig. 33.—Driving Heads for Steel Sheet-piles.

A pile-frame (Fig. 32) is often used with a double-acting pile hammer instead of a drop-hammer or monkey, since the pile-frame greatly assists maintaining alignment of the piling, while the rapid but light blows of a double-acting hammer are better for penetrating permeable granular soils, such as coarse sand and gravel. In this case the hammer is held in the leaders by back or side guides. Double-acting hammers are operated by steam or by compressed air, but the correct lubricant must be used in each case. The successful operation of a double-acting hammer depends on the whole weight of the hammer resting always on the head of the pile; otherwise the blow will be taken by the frame of the hammer, with
Fig. 34.—Steam Generator.

Fig. 35.—Diesel Hammer used with Hanging Leaders.
possibly disastrous results. For this reason the operation of some hammers cannot be started until the hammer is bearing on the pile. The hammer may be placed directly on the pile, or a driving head of one of the types shown in Fig. 33 to suit the section of the pile may be used; the type shown in the centre is used for drop hammers or single-acting hammers. When driving steel sheet-piles without a pile-frame, leg guides are commonly used in combination with temporary timber walings secured to temporary stakes or to the tops of the piles driven previously.

![Fig. 36.](image)

Some types of steel sheet-piles are supplied already interlocked in pairs to save time in driving, and helmets as shown in Fig. 33 are arranged to fit over the pair. This method enables rapid progress to be made and consumes only about three-quarters the total energy required to drive two piles separately; it also makes it easier to guide the piles. Where steel sheet-piles, particularly sections with thin webs, have to be driven in stiff cohesive soil, the bottom edges of the piles are occasionally reinforced by steel strips bolted or welded on so that the skin friction on the pile is reduced.

As an alternative to the ordinary steam boiler, particularly when sheet-piles are being driven by a steam-hammer hung from the jib of a diesel-powered crane
or excavator, the steam for the hammer can be made in a generator of the type shown in Fig. 34 in which the generator is placed some way from the pile-driving operation. Alternatively the generator can be mounted at the rear of a travelling crane.

In cases where the sheet-piles are being driven in inaccessible positions, or to avoid providing temporary stagings, hanging leaders suspended from a crane are used (Figs. 35 and 40). Such leaders often form part of a derrick with a hinged support at the end of the jib, and sometimes with a horizontal strut from the base of the crane. The crane may be mounted on crawler tracks or on a bogie on rails (Fig. 36).

**Extraction of Sheet-piles.**

Double-acting hammers can be used to withdraw steel piles as easily as they are driven if the hammer is inverted and fitted with an extracting attachment (Fig. 37a). Extraction may be more easy than driving in the case of undamaged piles in non-cohesive soils. A different type of jaw is used for extracting concrete and timber piles (Fig. 37b).
If a hole is provided in the web of a steel sheet-pile near the top, they can be extracted by an inverted hammer using either a flexible yoke (Fig. 38a), or a rigid yoke (Fig. 38b), the latter comprising parts that are easily replaceable in case continual battering should cause a fracture. The extracting gear should be properly annealed periodically.

When double-acting hammers are used for extracting piles, they are usually arranged to work as single-acting machines so that the piston is only driven upwards. Sometimes with the larger steam hammers, the exhaust from the low side is restricted so as to cushion the piston on its downward (idle) stroke when the hammer is inverted. Where large numbers of piles have to be withdrawn, an extractor (Fig. 39) is a suitable alternative to an inverted hammer.

The extraction of undamaged steel sheet-piles will normally be within the capabilities of an extractor suspended from a derrick (Fig. 3), even if the piling was driven some time previously. The exception would be for deep penetration in stiff clay. In this case, or if the piles are at all long or are damaged at the
lower ends, as when driven into soft or hard rock or forced past boulders, the extraction is likely to be difficult and costly as the resistance to start extraction may then approach or, exceptionally, exceed, 150 tons, and a strong A-frame or sheer-legs as well as a powerful extractor will be necessary. In the case of marine works this may be mounted on a barge of sufficient spare buoyancy; say, about double the expected starting pull. The extractor will require extra strong link straps and high-tensile bolts through the pile web and in the head of the extractor. The pull from the A-frame on the extractor will also need to be through a multi-part block-and-falls so that the lift is taken by up to a dozen parts of the hoisting cable to enable the use of a hoisting engine of reasonable size.

**Driving Timber and Concrete Sheet-piles.**

The driving of timber and concrete sheet-piles is similar to that of bearing piles but, as in nearly all cases the sheeting carries little if any vertical load, a moderate final set per blow will usually be satisfactory, and thus, due also to the usual comparative lightness of the piles, lighter pile-frames and smaller hammers may frequently be used. For timber and concrete sheeting a piling-frame is generally used with a drop-hammer, or sometimes with a single-acting steam-hammer in the case of heavy concrete sheet-piles.

With concrete sheet-piles a helmet and head-packing are always desirable
as for concrete bearing piles, but where the driving is easy a head-packing only of rope, sacking, or asbestos mats will be sufficient to save damage to the pile-head, but only so long as the pile drives plumb and is not being struck by eccentric blows.

Hard driving of reinforced concrete piles sometimes results in the main bars pushing through the top concrete cover of the pile, probably because the impacts during driving cause variations in the stress waves between the steel and the concrete so that the bond stress is overcome, which permits the main bars to be released from balancing the shrinkage stress in the concrete. Nevertheless, spalling of the concrete is reduced by having the tops of all the main bars at the same level at the head, and by proper maturing of the pile before driving, say one month for rapid-hardening Portland cement concrete and two months for ordinary Portland cement concrete. As a head packing, woven asbestos mats in several layers or plastic packings are more effective than sacking or sawdust in a sack, since besides avoiding charring with the heat generated in driving, the impact-absorbing characteristics of the woven asbestos remains sufficiently constant to enable more reliable estimates of the driving resistance by impact formulae.

For timber sheet-piles, unless the driving is easy, wrought-iron straps around the pile-heads are necessary to prevent "brooming" of the tops. Where it is desirable not to have to trim the pile-heads for a strap to be forced on, a small helmet fitting the pile-head is used. However, since damage to the pile may occur lower down also, the fall of the hammer should be limited more for sheet-piles than other timber piles.

BIBLIOGRAPHICAL REFERENCES (CHAPTER IV).

CHAPTER V
PRESSURES ON SHEET-PILE WALLS

Although many theories have been put forward to demonstrate the form and magnitude of the active and passive earth pressures on sheet-pile walls, they are generally influenced by the desire to obtain pressure diagrams which simplify the determination of the maximum bending moment and the minimum necessary depth of penetration. These simplifications are generally agreed to be justified, since the results obtained cannot be more reliable than the information available about the characteristics of the soil which vary in course of time and may vary along the length of the wall. For these reasons straight-line, or hydrostatic, pressure-distribution theories are generally used for the active and passive pressures.

Distribution of Earth Pressures.

The pressures on sheet-pile walls depend on several variable factors, and in particular on the depth of penetration. The probable distribution of the pressures in typical cases is shown in Fig. 41 (a) in the case of the piles being driven to the least depth practicable, and in Fig. 42 (a) in the case of the piles being

![Diagram](image)

**Fig. 41.**—Pressure on Sheetig with Minimum Penetration.

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so well driven that they are fixed rigidly in the ground. The corresponding simplified distribution generally assumed in design are shown in Figs. 41 (b) and 42 (b). Subsequently it will be seen that the assumptions, although apparently fairly seriously in error, are in fact safe for design, and facilitate the calculations which would otherwise be too complex for work other than that of exceptional importance.

The methods described in the following have the advantage of being adaptable to any acceptable theory of active and passive earth pressures provided it has a hydrostatic variation, and any desired values of active and passive pressures may be used. The effect of friction can also be taken into account if required. Active earth pressures may be based upon the theories of Rankine, Coulomb, or Bell. (5.1)* Research and observations of the behaviour of earth-retaining structures have shown that these theories give an untrue picture of the behaviour of earth pressures, unless they are qualified by additional conditions of the stiffness of the wall, the restraint of the base of the wall and the physical state of the soil. The formulæ of Professor C. F. Jenkins (5.2), (5.3) are dependent on the phenomenon of the dilation of granular soils, and are therefore only applicable for yielding walls and non-cohesive soils. Although the common wedge theory presumes a true plane of rupture, the form of the surface of rupture is generally a curve. Experience has shown also that the position at which this curve meets the surface of the ground behind the wall does not differ greatly, notwithstanding the com-

* References thus (1-1) refer to Bibliography on page 259.
paratively wide variations in the properties of the soil. Some investigators found it to be at a distance from the back of the wall equal to about 0.4 of the height of the wall and others have found it to be at a distance equal to half the height. However, some of these observations were on timbered cuttings and the form of the restraint to forward movement of the wall has a definite influence on the lateral pressure and the centre of pressure. Bell’s formulae is generally satisfactory for cohesive soils, provided the minimum angle of internal friction is

![Diagram](image)

Fig. 43.—Typical Failure of Cohesive Soil of Low Bearing Value.

taken as that of the soil at the greatest water-content likely and the reduction of lateral pressure provided by the cohesiveness is included only if it is not likely to be altered adversely in course of time.

Since cohesive soils normally have a relatively small angle of internal friction, and cohesion gives a constant addition to the shearing strength, the type of failure of cohesive soils is frequently that shown by Fig. 43 which is the type of failure first investigated by Fellenius and assumed by him and others subsequently to be either circular or some similar curve such as a logarithmic spiral. The figure is of interest here as indicating that for cohesive soils the stability of the whole structure is often governed by the properties of the soil completely outside the
areas of the wedges assumed in some theories. From the loading and the properties of the soil the surface of sliding (shearing) failure can be calculated, and often extends both deeper and over a greater length, as in the case of the failure of the Tun-ka-doo wharf at Shanghai, shown in Fig. 43 (b).

**Magnitude of Earth Pressures.**

For sheet-piling Rankine's formula is generally used, and friction is usually disregarded as an effect on the active pressure, thereby resulting in the simple expression for a total lateral force on unit length of a vertical wall if the surface of the ground behind the wall is horizontal,

$$P = \frac{wH^2}{2} \cdot \left(\frac{1 - \sin \phi}{1 + \sin \phi}\right)$$  \hspace{1cm} (I)

The resultant is assumed to act at one-third the height, and horizontally if the friction is ignored.

Coulomb's formula

$$P = \frac{wH^2}{2} \cdot \tan^2 \left(45^\circ - \frac{\phi}{2}\right)$$  \hspace{1cm} (IA)

although based upon slightly different assumptions, gives the same result in this case of horizontal surface behind the wall.

Bell adapted the Rankine formula to allow for cohesion and, adopting the same assumption of a triangular pressure diagram for that part of the lateral force which is not taken internally in the material by cohesion, the total pressure on a vertical surface becomes

$$P = \frac{wH^2}{2} \cdot \left(\frac{1 - \sin \phi}{1 + \sin \phi}\right) - 2C \sqrt{\frac{1 - \sin \phi}{1 + \sin \phi}}$$  \hspace{1cm} (2)

where $C$ is the cohesion, that is the shearing strength of the unloaded soil at the surface. However, as cohesion is a property of soils which may nearly disappear with increased water-content of the soil, Rankine's formula is generally used and cohesion is often ignored. On the other hand, the lateral active pressure has been shown to be normally less than that obtained by Rankine's formula if the wall deflects or tilts away slightly from the retained soil, the reduction being greater for soils with the higher angles of internal friction.

The formulae of Rankine, Coulomb, and Bell are given in Table VI. Reference should be made to Table VII and Fig. 44 for corresponding data in accordance with Jenkin's theory.

There is a fairly wide range of pressures against a wall for any given soil according to whether the wall is forced against the soil, resulting ultimately in developing the maximum passive resistance before being forced out of the way, or, in the other extreme, the wall moves or tilts away from the load and lower pressures than Rankine's active pressure are obtained. Rankine's results, it should be remembered, were obtained by considering the principal stresses in the soil and treating the soil itself as an incompressible solid. The assumption of this limiting condition of equilibrium is, however, modified when the soil can change its state by movement. Professor Jenkin investigated in great detail
### Table VI.—Earth Pressure Formulae

<table>
<thead>
<tr>
<th>Level top surface</th>
<th>Type of soil</th>
<th>Rankine</th>
<th>Coulomb</th>
<th>Bell</th>
<th>Jenkin</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Granular</td>
<td>Granular</td>
<td>Cohesive</td>
<td>Cohesieless granular</td>
<td></td>
</tr>
<tr>
<td>Active horizontal pressure at a depth ( h ) with level top surface</td>
<td>( wh \frac{(1 - \sin \phi)}{(1 + \sin \phi)} )</td>
<td>( wh \tan^2 \left( 45 - \frac{\phi}{2} \right) )</td>
<td>( wh \tan^2 \left( 45 - \frac{\phi}{2} \right) - 2C \tan \left( 45 - \frac{\phi}{2} \right) )</td>
<td>Given in table on Fig. 44</td>
<td></td>
</tr>
<tr>
<td>Maximum passive horizontal resistance at a depth ( d )</td>
<td>( wd \frac{(1 + \sin \phi)}{(1 - \sin \phi)} )</td>
<td>( wd \tan^2 \left( 45 + \frac{\phi}{2} \right) )</td>
<td>( wd \tan^2 \left( 45 + \frac{\phi}{2} \right) + 2C \tan \left( 45 + \frac{\phi}{2} \right) )</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>Maximum passive vertical resistance to downward pressure alongside at a depth ( d )</td>
<td>( wd \frac{(1 + \sin \phi)}{(1 - \sin \phi)} )</td>
<td>( wd \tan^2 \left( 45 + \frac{\phi}{2} \right) )</td>
<td>( wd \tan^2 \left( 45 + \frac{\phi}{2} \right) + 2C \tan \left( 45 + \frac{\phi}{2} \right) )</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>Active pressure at a depth ( h ) with surcharge angle ( i = \phi ) (see Fig. 44)</td>
<td>( whQ \frac{(1 - \sin \phi)}{(1 + \sin \phi)} )</td>
<td>Graphical method usually used</td>
<td>—</td>
<td>Given in table on Fig. 44</td>
<td></td>
</tr>
<tr>
<td>Maximum passive resistance at a depth ( d )</td>
<td>Use graphical method, say as Fig. 47</td>
<td>Ditto</td>
<td>—</td>
<td>—</td>
<td></td>
</tr>
</tbody>
</table>

### Table VII.—Comparison of Pressures due to Granular Soils.

<table>
<thead>
<tr>
<th>Angle of internal friction ( \phi ) (deg.)</th>
<th>Active horizontal pressure (lb. per sq. ft.) at a depth ( h ); ( w = 100 ) lb. per cu. ft. Level top surface</th>
<th>Maximum passive horizontal resistance (lb. per sq. ft.) to downward pressure at a depth ( d ); ( w = 100 ) lb. per cu. ft. Level top surface</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rankine and Coulomb</td>
<td>Jenkin</td>
</tr>
<tr>
<td>5</td>
<td>83.8</td>
<td>78.7</td>
</tr>
<tr>
<td>10</td>
<td>70.2</td>
<td>62.5</td>
</tr>
<tr>
<td>15</td>
<td>58.8</td>
<td>50.0</td>
</tr>
<tr>
<td>20</td>
<td>48.9</td>
<td>40.1</td>
</tr>
<tr>
<td>25</td>
<td>40.5</td>
<td>32.2</td>
</tr>
<tr>
<td>30</td>
<td>33.3</td>
<td>25.7</td>
</tr>
<tr>
<td>35</td>
<td>27.0</td>
<td>20.5</td>
</tr>
<tr>
<td>40</td>
<td>21.7</td>
<td>16.1</td>
</tr>
<tr>
<td>45</td>
<td>17.2</td>
<td>12.5</td>
</tr>
</tbody>
</table>

To obtain the pressure intensities from this table, multiply by \( h \) or \( d \) as the case may be, and also by the ratio of the density of the soil if different from the 100 lb. per cubic foot taken in the table.
Fig. 44.—Active Earth Pressure for Cohesionless Granular Soils with Surcharge (Jenkin).
the pressures of cohesionless granular soils and the small lateral movement of the wall necessary to conform to the assumptions. The factors for calculating the active pressure in accordance with Jenkin's theory are given in Table VI for comparison with Rankine's and Coulomb's pressures for various values of \( \phi \).

The centre of pressure assumed to be at \( \frac{H}{3} \) in Rankine's method is not justified by more recent research, since it varies with the state of packing of the soil and slight movement of the wall. Professor Jenkin recommended Weyrauch's proposal that the centre of pressure should be taken to be at \( 0.4H \) for horizontal surfaces and surcharge angles between \(-10\) deg. and \(+25\) deg. With a positive surcharge angle \( i \) approaching \( \phi \), the centre of pressure is higher and reaches \( 0.55H \) at \( i = \phi \). The variation of the centre of pressure with tilting of the wall is reproduced in Fig. 45 after Terzaghi.\(^{6,4}\)

In addition, the soil arches across deflected wall surfaces and concentrates, in the case of anchored sheeting walls, near the tie and bottom; and, if king piles are used, also in the horizontal direction. Danish engineers have long used reduction factors for the moments on sheet-pile walls, and a description of that method is given on page 93. Before considering this matter, it is necessary to note that a combination of data from different sources may result in ignoring qualifications to their use. Thus, assumptions for the active pressure found to lead to a safe design are as follows:

(a) Angle of internal friction taken as lowest likely during the life of the structure, which, if the wall is in flowing water, is usually the value when the soil is saturated; see Table VII.

(b) Active pressure by Rankine's formula.

(c) Centre of pressure at \( \frac{H}{3} \).

(d) Friction ignored for active pressure.

(e) Cohesion ignored.

(f) Reduction factor applied to moments in flexible walls.

The increased pressure, if water stands behind the wall higher than in front, must be allowed for, and a conservative estimate of the drainage rate must be taken. Thus in the most severe case for stresses in the wall, when there is no water in front, the slowness of the drainage may leave a fair height of water behind the wall.

Several of the foregoing assumptions are slightly incorrect, but in combination they are fairly satisfactory. For example, with granular soils like clean sand or gravel, (b) gives too high a value for the active pressure, but by assumption (c) the centre of pressure is usually too low if the wall does not tilt slightly about the base. If the wall is free to move slightly forward bodily when the earth loading comes on it, the centre of pressure will pass through, or perhaps stay, at the stage where it is \( 0.4H \), but the slight movement will ensure the active pressure coefficient falling below Rankine's value and for granular soils may then be taken from Professor Jenkin's values as given in Table VI and in the table on Fig. 44.

For rigid walls which become loaded by pumped filling, the hydrostatic pressure added temporarily must be allowed for, and the active soil pressure
will approach that of the natural state of the soil, which will exceed Rankine's values.

Cohesion is too variable a property to take into account in a method of design for sheet-pile walls unless there is protection of the soil from seasonal variations in water-content and perfect drainage, a rare combination. It is reasonable, however, for the granular materials usually chosen for filling behind these walls, and provided the soil is cohesionless, to use active pressures obtained from Professor Jenkin's coefficients. This applies particularly to walls with a

surcharge, either negative or positive, as the pressures by Rankine's formula for positive surcharge are known to be excessive. The centre of pressure should then be taken as \(0.4H\), or higher for positive surcharge, as the acceptance of yield to reduce the active pressure is seen by Fig. 45 to involve also a probable rise in the centre of pressure.

When the surface of the backfill inclines away upward from the wall, giving positive surcharge, the active pressure can be found from what is usually known as Rehann's construction, the method being essentially also that proposed earlier by Poncelet. However, it is simpler in this case, and particularly for cases where the back of the wall itself also slopes, to use Jenkin's tables \(^{(5.3)}\) giving directly the coefficients for the active pressure; the coefficients can be adjusted to correspond to Rankine's formula for active pressure if they are multiplied by the factor \(\frac{\rho_a \text{ (by Rankine)}}{\rho_a \text{ (by Jenkin)}}\) both taken for \(i = 0\) and for the correct value of \(\phi\).
PRESSURES ON SHEET-PILE WALLS

It is not from choice that a sheet-pile wall would be used for retaining clay and, unless it can be maintained at its natural water-content or slightly drier, not only will the internal friction drop occasionally to very low values but the cohesion component of the shearing strength does the same, and together these result in great increase in the active pressure. In addition, since superimposed loading, when the clay is in a plastic state, obviously increases the active horizontal pressure in closely the same way as a liquid, a sheet-pile wall should wherever possible be backed by granular material extending beyond the plane of rupture, and the clay beyond that line should be drained or maintained close to the original water-content. Drainage of the surface to some distance behind the wall may accomplish this.

Properties of Cohesive Soils.

It is sometimes necessary to take account of cohesion in the earth pressures of cohesive soils after making due allowance for any increase in water-content and consequent reduction in the values of $\phi$ and $C$, since the soil properties when $C$ is ignored may otherwise lead to unduly heavy construction. The values of the coefficients to Bell’s formula given in Table VIII will then be found convenient for obtaining the active and passive pressure intensities at any depths desired.

Typical values of $C$ are given by Bell in his original paper (8.1) varying from 45 lb. per square foot for very soft clay to 3600 lb. per square foot for very stiff boulder clay; the values for $\phi$ are 0 deg. and 16 deg. respectively. If it is considered necessary to take account of cohesive properties in the design, the actual properties of the cohesive soil concerned in the proposed work should be ascertained and possible deterioration taken into account.

Table VIII.—Values of Coefficients in Bell’s Formula for Various Values of Angle of Internal Friction.

<table>
<thead>
<tr>
<th>$\phi$ (deg.)</th>
<th>$\tan^2 \left(\frac{45 + \phi}{2}\right)$</th>
<th>$\tan^2 \left(\frac{45 - \phi}{2}\right)$</th>
<th>$\tan^2 \left(\frac{45 + \phi}{2}\right)$</th>
<th>$\tan^2 \left(\frac{45 + \phi}{2}\right)$</th>
<th>$\tan^2 \left(\frac{45 + \phi}{2}\right)$</th>
<th>$\tan^2 \left(\frac{45 - \phi}{2}\right)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.072</td>
<td>0.954</td>
<td>1.035</td>
<td>1.018</td>
<td>0.9657</td>
<td>0.9827</td>
</tr>
<tr>
<td>2</td>
<td>1.150</td>
<td>1.110</td>
<td>1.072</td>
<td>1.035</td>
<td>0.9326</td>
<td>0.9657</td>
</tr>
<tr>
<td>3</td>
<td>1.233</td>
<td>1.170</td>
<td>1.111</td>
<td>1.034</td>
<td>0.8905</td>
<td>0.9490</td>
</tr>
<tr>
<td>5</td>
<td>1.416</td>
<td>1.300</td>
<td>1.191</td>
<td>1.091</td>
<td>0.8397</td>
<td>0.9163</td>
</tr>
<tr>
<td>7</td>
<td>1.630</td>
<td>1.444</td>
<td>1.278</td>
<td>1.130</td>
<td>0.7828</td>
<td>0.8847</td>
</tr>
<tr>
<td>10</td>
<td>2.017</td>
<td>1.693</td>
<td>1.420</td>
<td>1.192</td>
<td>0.7041</td>
<td>0.8391</td>
</tr>
<tr>
<td>15</td>
<td>2.885</td>
<td>2.213</td>
<td>1.698</td>
<td>1.303</td>
<td>0.5875</td>
<td>0.7673</td>
</tr>
<tr>
<td>20</td>
<td>4.160</td>
<td>2.913</td>
<td>2.040</td>
<td>1.428</td>
<td>0.4903</td>
<td>0.7002</td>
</tr>
</tbody>
</table>

Laboratory tests of the shearing strength of clay can be supplemented by making Bell’s test on the site. If a steel ball of $\frac{1}{8}$ in. diameter is dropped 24 in. the following data apply:

<table>
<thead>
<tr>
<th>Diameter of Impression (inch)</th>
<th>1.5</th>
<th>1.3</th>
<th>1.1</th>
<th>1.0</th>
<th>0.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion (ton per square foot)</td>
<td>0.2</td>
<td>0.3</td>
<td>0.5</td>
<td>0.7</td>
<td>1.6</td>
</tr>
</tbody>
</table>

This test should be made only on a level surface of clay which has been newly exposed. See also page 5.
Loading imposed on Ground behind Wall.

A uniformly-distributed load imposed on the ground retained by the wall may be treated as equivalent to an increased depth of soil; thus an imposed load of 240 lb. per square foot is represented, for soil weighing 120 lb. per cubic foot, by calculating the pressures as if $H$ were increased by $\frac{240}{120} = 2$ ft.

To ascertain the pressures in the soil due to loads concentrated on the surface of the ground behind the wall, methods based on assumed angles of dispersion should be considered as inapplicable, except perhaps when the soil is excessively stressed. The application of Boussinesq's equations leads to serious underestimation of the stresses when the load is near the wall due to the effects of the boundary conditions compared with a semi-infinite continuum. Assuming the concentrated load is not great enough to cause the wall to yield so much that fracture planes develop within the backfill, then the experimental work of Gerber at Zürich and Spangler in America enables reasonable results to be obtained by adaptation of Boussinesq's equations.

The accuracy of the results depends on Poisson's ratio for the soil, but taking it as 0.5 (which it cannot exceed in any elastic solid) the empirical formula put forward by Spangler (5,6) for the normal unit pressure ($\varphi$) on the back of the wall at a point whose co-ordinates are $x$, $y$, and $z$ (Fig. 46) is

$$\varphi(= h_c \text{ in Fig. 47}) = \frac{KP}{x^{0.25} \cdot \frac{x^2 z}{R^8}}$$

in which $P$ is the applied load and $K$ is an empirical constant which in the experiments was $1.1$ for average results and $1.3$ for the maximum stresses recorded.

A comparison of experimental results is given in Fig. 47 and since, within limits, the principle of superimposition can be applied, the effect of a series of point or rolling loads can be obtained without difficulty was as shown by the writer (5,6) shortly after Spangler's first experiments on this subject.
PRESSURES ON SHEET-PILE WALLS

The lateral pressure due to a load concentrated on a continuous strip of ground immediately behind the top of a stiff cantilevered wall, the filling behind which was crushed stone graded from \( \frac{1}{16} \) in. to \( \frac{1}{4} \) in., was investigated in Sweden by Jansson and others.\(^5,7\) It was found that the pressure approached maximum about equal to that calculated for a fully-distributed surcharge using Rankine's formula, when the width of the strip was 0.9 times the height of the wall. About two-thirds of the pressure remained after removal of the load. While the results would be different with other types of filling, a similar persistence of much of the pressure from rolling loads, such as cranes or railway wagons, is likely to occur, and unless the tracks are close to the wall, which they should never be unless supported independently, a fair approximation is to distribute the rolling loads along the entire length of the vehicle or train. This strip of equivalent load, which should not be confused with that referred to in the foregoing, may then be assumed to act as in Fig. 48, where \( P = W \) times the active-pressure coefficient of the soil.

Fig. 47.—Lateral Pressure on a Retaining Wall due to a Concentrated Load on the Surface of the Backfill.
Submerged Earth.

If \( w_e \) is the weight per cubic foot of the dry soil, then when it is submerged the lateral pressure will be the sum of the hydrostatic pressure \( w \) where \( h \) is the depth from the top of the water (which is not necessarily the full depth \( H \)) plus the net active horizontal pressure of the soil in water. For the latter the unit weight of the soil is reduced by the buoyancy of the net solids and the active pressure obtained, using the angle of internal friction in water, which may be taken at 5 deg. to 10 deg. less than when dry if any movement of the water is possible. Thus by Rankine's method, assuming fresh water and that \( \phi' \) is the angle of internal friction in water, then at a depth \( h \) below a common water and earth surface

\[
\phi_a = 62.5h + \left[w_e - \left(\frac{100 - v}{100}\right)62.5\right]\left(\frac{1 - \sin \phi'}{1 + \sin \phi'}\right)h.
\]

where \( v \) is the percentage of voids in the soil and \( w_e \) is the weight per cubic foot of the dry soil. That this pressure is actually developed is seldom now disputed. Where the percentage of voids is not known it may be fairly closely obtained by remembering that the density of the particles of soil seldom varies much from that of quartz (167 lb. per cubic foot), while for sedimentary granular soils in their natural condition the assumption of a porosity of 40 per cent. of voids is a rough but reasonable approximation. The description "dry" in the foregoing, means quite dry, not damp, soil.

Properties of Soils.

The majority of sedimentary soils have varying densities according to the degree of compaction and, in the case of clays, according to the moisture content.
and the pressure to which they have been subjected. Near the surface soils are also subject to changes due to climatic conditions, and under water to disturbance also. Because of these factors the weight per cubic foot cannot be given more than approximately for each type of soil. The angle of repose is practically the same as the angle of internal friction (\( \phi \)), difficulties in measurement of the latter being one cause of reported differences, so that if the angle of repose is taken on the middle part of a slope it is probably a good measure of the angle of internal friction. *Table IX* gives typical values of density and angles of repose for a variety of soils.

**Table IX.—Density and Angle of Repose of Soils.**

Figures in brackets give the percentage of voids for the density shown.

<table>
<thead>
<tr>
<th>Material</th>
<th>Density when dry (lb. per cubic foot)</th>
<th>Angle of repose (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loose (dry)</td>
<td>Compacted or natural deposit (damp)</td>
</tr>
<tr>
<td>Coarse sand (well graded)</td>
<td>100 (40)</td>
<td>115 (31)</td>
</tr>
<tr>
<td>Fine sand</td>
<td>90 (46)</td>
<td>105 (37)</td>
</tr>
<tr>
<td>Gravel</td>
<td>110 (34)</td>
<td>120 (28)</td>
</tr>
<tr>
<td>Shingle</td>
<td>95 (43)</td>
<td>100 (40)</td>
</tr>
<tr>
<td>Brick hardcore</td>
<td>80</td>
<td>110</td>
</tr>
<tr>
<td>Quarry waste</td>
<td>90-100</td>
<td>100-120</td>
</tr>
<tr>
<td>Broken rock</td>
<td>90 (46)</td>
<td>110 (34)</td>
</tr>
<tr>
<td>Ashes</td>
<td>40</td>
<td>50</td>
</tr>
<tr>
<td>River mud</td>
<td>—</td>
<td>90</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>—</td>
<td>110</td>
</tr>
<tr>
<td>Soft clay</td>
<td>—</td>
<td>95</td>
</tr>
<tr>
<td>Medium clay</td>
<td>—</td>
<td>105</td>
</tr>
<tr>
<td>Very stiff clay</td>
<td>—</td>
<td>115</td>
</tr>
</tbody>
</table>

*Angle of internal friction. Cohesion to be allowed for separately, say, by Bell's formula.*

When filling is saturated but not inundated, such as immediately after the tide has receded, the excess water temporarily reduces the internal friction so that the angle \( \phi \), according to some tests on sand and gravel, is some 15 deg. lower than for the same material dry. It is, however, reasonable to expect that this effect, which is large in model tests, only affects consecutive fractions of the full depth of the fill. Experience indicates that present design methods may safely be used unaltered, provided it is appreciated that the apparent factor of safety is thus encroached upon, except where there are conservative assumptions in other respects.

It is worth noting that with the usual siliceous soils—sands, gravels, and shingle—the total lateral pressure in still water, including the water pressure, varies little, the effect of variation in percentage of voids and reduced weight of the soil by buoyancy being approximately cancelled by the corresponding change in the value of \( \phi \). Impermeable soils will seldom be subject to buoyancy, or if they are it will be because the soil structure has been broken down, for example, by driving piles. In both cases it may be expected that the lateral pressure below the water level will be not less than that of the same soil above water level.
Passive Resistance of Earth.

As for active pressures, the passive resistance of the soil could be based upon one of the principal theories of earth pressure.

Bell’s formula is seldom used since the bottom of the wall is generally in the bed of a waterway which may be exposed at low tide so that, even if the soil in front of the wall has cohesive properties, they are susceptible to variation and it is best to ignore cohesion.

Rankine’s formula is generally used, but there are various ways of doing so. It is not only necessary for the penetration to be sufficient for the resistance in the front of the wall, added to the tension in the tie, if any, to balance the horizontal forces due to active earth pressure, but an excess to provide a factor of safety is necessary. Sometimes also some restraint is obtained at the base by added penetration, thereby reducing the moments in the wall above. If both cohesion and wall friction are disregarded in calculating the passive resistance, the penetration as obtained from Rankine’s formula will normally be sufficient to give a reasonable factor of safety. The Danish rules (page 95) place greater emphasis on a substantial penetration by requiring that the calculated passive resistance of the ground in front of the wall at the bottom shall be double the active pressure to be resisted. Modifications of this rule apply if the piles are spaced apart. The apparent contradictions in these two methods are not as great as they seem. If the assistance of friction on the wall to increase the passive resistance is not taken into account, it is seen from Fig. 49 that a substantial increase in the passive resistance has been ignored; this diagram is based on Coulomb’s formula, which is

\[
\phi_p = wh \frac{\sin^2 (\theta - \phi)}{\sin^2 \theta \sin (\theta + \mu) \left( I - \sqrt{\frac{\sin (\mu + \phi) \sin (\phi - i)}{\sin (\theta + \mu) \sin (\theta - i)}} \right)^2} \quad (5a)
\]

in which \( \phi \) is the angle of internal friction of the soil, \( \mu \) is the greatest angle of friction between the soil and the wall, \( i \) is the angle of the slope of the ground behind the wall, \( w \) is the density of the soil, \( \phi_p \) is the intensity of passive pressure at depth \( h \), and \( \theta \) is the angle between the front face of the wall and the horizontal. In the particular case of level filling behind a vertical wall, \( i = 0, \theta = 90 \ deg. \)

\[
\phi_p = wh \frac{\cos^2 \phi}{\cos \mu \left( I - \sqrt{\frac{\sin (\mu + \phi) \sin \phi}{\cos \mu}} \right)^2} \quad (5b)
\]

The greatest angles recommended by Mr. S. Packshaw (5,8) for submerged soil are \( \frac{1}{4} \phi \) for timber and concrete piles, \( \frac{1}{4} \phi \) to \( \frac{1}{3} \phi \) for uncoated steel piles, and \( \phi \) for steel piles coated with tar. By using Fig. 49, an angle of \( \frac{1}{4} \phi \) cannot be inadvertently adopted if \( \mu \) is less than \( \frac{1}{2} \phi \). It is seen, however, from the diagram in Fig. 49 that the additional passive resistance due to friction is largely dependent on the angle of internal friction of the soil. The angle of friction between the wall and the soil cannot exceed the smaller of the angles \( \phi \) and \( \mu \). For wet soils, the values of \( \mu \) recommended by the writer are \( \tan^{-1} 0.3 \) (17 deg.) for steel sheet-
piles in sand, and \( \tan^{-1} 0.4 \) (22 deg.) for smooth concrete piles in sand or gravel. For further remarks on the friction between the soil and the piles see page 86.

In the Danish rules it is assumed that the direction of the passive earth pressure acts upwards at an angle of \( \frac{\phi}{2} \) to the normal to the wall, and cohesion is ignored as recommended.

The results by any method will depend largely upon the correct estimation of \( \phi \) for the soil in front of the bottom of the wall; cohesion should not be depended upon to provide part of the factor of safety. The best method for passive resistance is possibly the following:

1. Passive earth pressure by Rankine's formula.
2. Multiply this resistance by the coefficient \( M \) obtained from Fig. 49 to allow for friction.
3. For cantilever walls multiply the penetration thus determined by \( \sqrt{2} \), thereby giving a factor of safety of about 2 against tilting forward of the foot.

Fig. 49.—Effect of Friction in Increasing the Passive Resistance of Soil.
of the wall, ignoring any assistance from cohesion. For walls with ties use the same increase of penetration if driven only to obtain simple support at the bottom or about 10 per cent. when the penetration is to full restraint.

Full allowance must, of course, be made for any dredging that may be done in front of the wall and for the effect of scour and for the reduced passive resistance if the channel bed shelves downwards away from the sheet-piling towards the centre of the waterway as considered in the following.

**Passive Resistance with "Negative Surcharge".**

A graphical method of obtaining passive earth pressure when the soil in front recedes downwards from the face of the sheet-piling, has been deduced graphically

![Graphical Method for Determining the Passive Resistance of Negative Surcharge of Slope of Channel Bed.](image)

\[ P = \Delta \text{FGH} \times \text{UNIT SOIL WEIGHT.} \]

Fig. 50.—Graphical Method for Determining the Passive Resistance of Negative Surcharge of Slope of Channel Bed.

by Professor Andersen (5,9) and is an extension of Poncelet's construction. The method, which is applicable to cohesionless soils, is shown in Fig. 50.

The total active pressure of a cohesionless soil behind a wall can be expressed graphically as the area of a right-angled triangle multiplied by the density of the soil. This idea can be extended to the determination of the passive pressure in front of a wall which, according to Coulomb, is given by

\[ P = \frac{w}{2} \left[ \frac{h \cos \phi}{1 - \sqrt{\sin \phi (\sin \phi - \cos \phi \tan i)}} \right]^2. \quad (6) \]

in which the notation is as for formula (5a) except that in this case \( i \) is the angle between the horizontal and the surface of the ground in front of the wall.

The construction is as follows. Locate the point of intersection \( C \) between the extension \( AG \) of the surface of the bank in front of the wall and a line through the bottom \( B \) of the sheet-piles at an angle to the horizontal equal to the angle of internal friction \( \phi \). With \( BC \) as a diameter, draw a semicircle and intersect it at \( D \) with a line from \( A \) perpendicular to \( BC \). Make \( BF \) equal to \( BD \) and
PRESSURES ON SHEET-PILE WALLS

erect GF normal to BC. If FH is made equal to FG, then the area of triangle GFH multiplied by the density \( w \) will be equal to the passive pressure \( P \). The distance FC is the sum of FB and BC, that is, the sum of BD and BC. Therefore

\[
FC = \frac{h}{\sin \phi - \cos \phi \tan i} + \frac{h \sqrt{\sin \phi}}{\sqrt{\sin \phi - \cos \phi \tan i}}
\]

\[
= h \frac{1 + \sqrt{\sin \phi \left( \sin \phi - \cos \phi \tan \frac{i}{\sin \phi - \cos \phi \tan \phi} \right)}}{\cos \phi + \sin \phi \tan \phi}
\]

(7a)

But \( FG = FC \tan (\phi - i) \), from which

\[
FG = h \frac{1 + \sqrt{\sin \phi \left( \sin \phi - \cos \phi \tan \frac{i}{\cos \phi + \sin \phi \tan \phi} \right)}}{\cos \phi + \sin \phi \tan \phi}
\]

(7b)

If the numerator and denominator in (7b) are multiplied by

\[
1 - \sqrt{\sin \phi \left( \sin \phi - \cos \phi \tan \frac{i}{\cos \phi + \sin \phi \tan \phi} \right)}
\]

this expression becomes

\[
FG = \frac{h \cos \phi}{1 - \sqrt{\sin \phi \left( \sin \phi - \cos \phi \tan \frac{i}{\cos \phi + \sin \phi \tan \phi} \right)} \sin \phi}
\]

(7c)

which is the term between brackets in formula (4b).

Hence \( P = \frac{w}{2} (FG)^2 = \frac{w}{2} \times (FG \times FH) = w \times \text{area of the triangle} \ GFH. \)

When the channel bed is not horizontal, but slopes downward away from the wall as a straight slope as in the preceding case, it is possible by approximations, consistent with the probable accuracy of other earth-pressure calculations, to obtain the equivalent exposed height of the wall and the increased penetration necessary. Thus if the bed slopes at \( i \) deg. from the horizontal, instead of the actual exposed height \( H \) the equivalent exposed height \( H' \) is as given in Table X in which \( D \) is the penetration calculated for the actual height for a horizontal bed.

<table>
<thead>
<tr>
<th>Angle</th>
<th>Values of ( H' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi = 30 ) deg.</td>
<td>( \phi = 25 ) deg.</td>
</tr>
<tr>
<td>5 deg.</td>
<td>( H + 0.11D )</td>
</tr>
<tr>
<td>10 deg.</td>
<td>( H + 0.22D )</td>
</tr>
<tr>
<td>15 deg.</td>
<td>( H + 0.33D )</td>
</tr>
<tr>
<td>20 deg.</td>
<td>( H + 0.45D )</td>
</tr>
<tr>
<td>25 deg.</td>
<td>( H + 0.57D )</td>
</tr>
<tr>
<td>30 deg.</td>
<td>( H + 0.71D )</td>
</tr>
</tbody>
</table>

Friction on Sheet-pile Walls.

Friction between the soil and sheet-pile walls does not greatly reduce the resultant active pressure behind the wall and should be ignored but, as shown in the foregoing, it may assist considerably the passive resistance of the soil.
The coefficient of friction that obtains under the most unfavourable conditions, which is when the soil is wet, is the determining factor.

The coefficient of friction may be assumed to be 0.3 to 0.6 of the horizontal pressure, the lower value applying to wet sand and the higher to dry sand. Tests, made by the writer, of the friction between sand and fairly smooth concrete, show that for wet sand the coefficient cannot safely be taken higher than 0.4. While this value may appear low, the desirability of a conservative estimate is apparent.

![Diagram](a)  ![Diagram](b)  ![Diagram](c)

Fig. 51.

Some authorities, such as Franzius, basing their opinion on experiments, adopt multipliers increasing the Rankine value to give the estimated actual passive resistance. A multiplier of two is often used in the U.S.A., but in the following examples the Rankine value is only increased for friction as shown by Fig. 49, and no other multiplier is used.

Roughly, and without encroaching on the safety of Rankine's theory, friction may be included for by multiplying the passive resistance value

$$p_p = wh\left(\frac{1 + \sin \phi}{1 - \sin \phi}\right)$$

by 1.5 for soils in which $\phi$ exceeds 20 deg.; having regard to current theories this is conservative.

The term $\phi$ represents whichever internal angle of friction of the soil is applicable that for dry soil or that for wet soil. Theoretically for perfectly still water they are identical, but movement of water within the soil, particularly seepage, greatly affects this angle (see Table VII).

If fine granular soil is wet from a receding tide, the angle of internal friction is lower than in any other condition of dryness or wetness due, it is thought, to lubrication by the excess water temporarily retained.

**Tabulated Pressures and Bearing Capacities.**

Caquot and Kerisel published\(^{(5,10)}\) results of analyses of earth pressures, being essentially an extension of the work of Rankine and Boussinesq in the
nineteenth century and taking account also of the work of Coulomb and Resal; it should be noted that the work of Jenkin (5.2), (5.3) also followed and improved on that of Resal. More recently, Caquot and Kerisel published tables (11.11) of active pressures, passive resistances and bearing capacities of cohesive and non-cohesive soils. Walls having various inclinations and surcharges are included, the angle of friction on the wall varying between $\phi$ and nil and with and without imposed loads. The following extracts show the scope of the tables, in which $\phi$ is the angle of internal friction and $\bar{\omega}$ is the density of the soil.

**Active Pressure of Granular Soil without Imposed Load.**—Referring to Fig. 51a, the angles $\omega$, $\alpha$ and $\beta$ are positive when in the directions shown. The values of $\phi$ for $\beta = 0$, $\omega = 0$ are given as follows.

\[
\begin{array}{|c|c|c|c|c|}
\hline
\phi & 10 & 20 & 30 & 40 & 50 \text{ deg.} \\
\hline
\phi & 0.649 & 0.440 & 0.308 & 0.219 & 0.155 \text{ when } \frac{\alpha}{\phi} = \mp 1 \\
\phi & 0.704 & 0.490 & 0.333 & 0.217 & 0.133 \text{ when } \frac{\alpha}{\phi} = 0 \\
\hline
\end{array}
\]

\[P = \frac{1}{2}bl^2\bar{\omega} \text{ applied at the lower third-point of the wall.}\]

**Active Pressure of Cohesive Soil without Imposed Load.**—Fig. 51a applies. If the cohesion is $C$, let $H = \frac{C}{\tan \phi}$. The pressure is the resultant of three terms $P_1$, $P_2$ and $P_3$, such that

\[P_1 = \frac{1}{2}bl^2\bar{\omega} \text{ as if the soil were cohesionless and acts at the lower third-point at an angle } \alpha.\]

\[P_2 = \frac{Hl}{\tan \left(\frac{\pi}{4} + \frac{\phi}{2}\right)} \times 10^{-0.007588} \tan \phi \text{ in which } \delta = \frac{\pi}{4} - \frac{\phi}{2} + \bar{\omega} - \beta, \text{ and acts at the mid-height of the wall at an angle } \alpha = + \phi.\]

\[P_3 = H\bar{\omega}l \text{ acting normal to the wall at the mid-height.}\]

**Passive Resistance of Granular Soil without Imposed Load.**—Referring to Fig. 51b, the angles $\bar{\omega}$ and $\beta$ are positive and $\alpha$ is negative when in the directions shown. The values for $b$ and of its horizontal component $n$ for $\bar{\omega} = 0$, $\beta = 0$, and $\alpha = - \phi$ are as follows.

\[
\begin{array}{|c|c|c|c|c|}
\hline
\phi & 10 & 20 & 30 & 40 & 50 \text{ deg.} \\
\hline
b & 1.64 & 3.51 & 6.42 & 10.5 & 17.5 \\
n & 1.62 & 2.83 & 5.56 & 13.4 & 47.8 \\
\hline
\end{array}
\]

\[B = \frac{1}{2}bl^2\bar{\omega} \text{ and } N = \frac{1}{2}nl^2\bar{\omega} \text{ and act at the lower third-point of the wall.}\]
For other values of $\phi$, the resistances in the foregoing are multiplied by a coefficient as follows.

<table>
<thead>
<tr>
<th>$\phi$ (deg.)</th>
<th>Values of $\alpha$</th>
<th>nil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$-0.6\phi$</td>
<td>$-0.3\phi$</td>
</tr>
<tr>
<td>10</td>
<td>0.962</td>
<td>0.912</td>
</tr>
<tr>
<td>20</td>
<td>0.901</td>
<td>0.787</td>
</tr>
<tr>
<td>30</td>
<td>0.811</td>
<td>0.627</td>
</tr>
<tr>
<td>40</td>
<td>0.682</td>
<td>0.439</td>
</tr>
<tr>
<td>50</td>
<td>0.506</td>
<td>0.242</td>
</tr>
</tbody>
</table>

Passive Resistance of Cohesive Soil without Imposed Load.—Fig. 51b applies. As before, the cohesion $= C$ and $H = \frac{C}{\tan \phi}$. The resistance is the resultant of three terms $B_1$, $B_2$ and $B_3$.

$B_1 = \frac{1}{2}l_p l_w \overline{\omega}$ which is $\frac{1}{2}l_p l_w \overline{\omega}$ in which $\rho_1$ is obtained and acts as in the foregoing section.

$B_2 = Hl \tan \left(\frac{\pi}{4} + \frac{\phi}{2}\right) \times 10^{0.00758 \times 2\delta \tan \phi}$ in which $\delta = \frac{\pi}{4} + \frac{\phi}{2} + \omega - \beta$, and acts at the middle of the wall at angle $\alpha = -\phi$.

$B_3 = -Hl$ acting normal to the wall at mid-height.

Bearing Capacity of Granular and Cohesive Soils.—The ultimate resistance is the arithmetical sum of three terms $r_1$, $r_2$ and $r_3$ for all soils, together with the terms $r_4$ if the soil below the bottom is cohesive and $r_5$ if the soil in height $h$ is cohesive, all multiplied by the area on plan of the base. The evaluation of the terms are as follows. Fig. 51c applies.

$r_1$ (due to soil under the base) = $\overline{\omega}l_s l_1$ in which

$$s_1 = 0.192 \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2}\right) \left(3.15 + 0.0005 \tan \phi - 1\right).$$

$r_2$ (due to soil above the level of the base) = $h \overline{\omega} s_2 s_2'$ in which

$$s_2 = \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2}\right) \tan \phi$$

and $s_2' = 1 + 0.32 \tan^2 \phi'$, where $\overline{\omega}'$ is the mean density of the soil in height $h$ and $\theta'$ is the mean angle of internal friction in height $h$.

$r_3$ (due to lateral friction) = $\overline{\omega} \frac{h^2}{l_2} s_3'$ in which $s_3' = \tan \phi e^{\frac{\pi}{2} + \phi'(4 + \tan \phi')}$;

$r_3$ reduces, or even changes sign, if the surface is loaded later as such loading may change the direction of the friction on the sides of the piles.

$r_4$ (for cohesive soils only) = $H(s_2 - 1)$ in which $H = \frac{C}{\tan \phi}$.

$r_5$ (for cohesive soils only) = $C' \frac{h}{l} s_2'$ in which $C'$ = mean cohesion of the soil in height $h$, and $s_2' = (1 + \sin \phi') e^{\frac{\pi}{2} + \phi'} \tan \phi'$. 
PRESSURES ON SHEET-PILE WALLS

Values of the terms $s_1$, $s_2$, etc., are as follows.

<table>
<thead>
<tr>
<th>$\phi$ or $\phi'$</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50 deg.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_1$</td>
<td>0.336</td>
<td>1.66</td>
<td>7.39</td>
<td>39.3</td>
<td>327</td>
</tr>
<tr>
<td>$s_2$</td>
<td>2.47</td>
<td>6.40</td>
<td>18.4</td>
<td>64.2</td>
<td>319</td>
</tr>
<tr>
<td>$s_3$</td>
<td>1.01</td>
<td>1.04</td>
<td>1.11</td>
<td>1.22</td>
<td>1.45</td>
</tr>
<tr>
<td>$s_4$</td>
<td>0.285</td>
<td>1.03</td>
<td>3.21</td>
<td>11.3</td>
<td>56.9</td>
</tr>
<tr>
<td>$s_5$</td>
<td>1.60</td>
<td>2.70</td>
<td>5.01</td>
<td>10.4</td>
<td>32.3</td>
</tr>
</tbody>
</table>

**Resistance to Withdrawal.**

The bearing capacities given in the foregoing apply to withdrawal of piles except that $r_1$, $r_2$ and $r_4$ are each equal to zero and in the expression for $r_3$ the term $s'_2$ is modified due to the mean angle of friction being inclined downwards at angle $+\phi'$ and becomes $- \tan \phi'[(1 - \sin \phi')^2 + 0.14(1 - \cos 4\phi')]$; the modified values of $s'_2$ are as follows.

<table>
<thead>
<tr>
<th>$\phi'$</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50 deg.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s'_2$</td>
<td>0.126</td>
<td>0.200</td>
<td>0.266</td>
<td>0.335</td>
<td>0.389</td>
</tr>
</tbody>
</table>

**Safe Bearing Pressure on Cohesive Soils.**

From the least value expected of the cohesion, if subject to future variation, obtained from laboratory tests or from site tests or both as described on page 4, the approximate safe bearing pressure, with a factor of safety of three, is as follows, $C$ being the cohesive strength in tons per square foot.

<table>
<thead>
<tr>
<th>$\phi$ (degrees)</th>
<th>Bearing pressure in tons per square foot</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strip</td>
</tr>
<tr>
<td>Clay</td>
<td>0</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>4</td>
</tr>
</tbody>
</table>

Settlement is not considered in this evaluation and where desirable that settlement shall be very small, the pressure may need to be reduced or the piles taken to firmer soil, particularly where the pressure bulbs in the sub-soil coalesce with each other or with those of other structures.\(^{(5.12)}\), \(^{(5.13)}\), \(^{(5.14)}\)

**NOTE.—**For Bibliographical References for Chapter V see page 259.
CHAPTER VI

DESIGN OF SHEET-PILE WALLS

Sheet-pile walls are of two principal types, namely, simple cantilever walls, and walls which are tied back to anchors behind the wall.

Cantilevered Sheet-pile Walls.

The simplest case of a sheet-pile wall is where sheet-piles are driven sufficiently deeply to retain the earth without any support at the top of the piles. However, as the depth of penetration necessary is fairly large compared with the height of the bank retained, such a wall is restricted mostly to cases where the height of the bank is small or local circumstances prevent anchorage of the top of the wall.

Fig. 52.—Cantilevered Sheet Retaining Walls.

The method of calculation in the following is dealt with as the general case for this type of wall where the soil does not vary much in the depth, so that average characteristics can be employed with sufficient accuracy. It is necessary to determine the penetration $D$ required such that the forces balance as well as the moments. The pressures may be assumed to be of the form shown in Figs. 41(a) and 52.

Equating horizontal forces and multiplying by two,

$$\frac{p_a(H + D)}{D} - \frac{p_p}{D} (H + 2D) X = 0$$

Therefore

$$X = \frac{\frac{p_p}{D} (H + D) - \frac{p_a}{D} (H + D)^2}{(\frac{p_p}{D} - \frac{p_a}{D}) (H + 2D)}$$

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Equating moments of these values about the bottom of the sheet-piles and multiplying by three,

\[ \rho_a(H + D)^3 - \rho_pD^3 + (\rho_p - \rho_a)(H + 2D)X^2 = 0 \]  

(9a)

To simplify, use a factor \( F \) such that \( D = FH \). Substituting for \( D \) in formula (9),

\[ X = \frac{\rho_pF^2H^2 - \rho_a(H + FH)^2}{(\rho_p - \rho_a)(H + 2FH)} \]

Substituting for \( D \) and \( X \) in formula (9a),

\[ \rho_a(H + FH)^3 - \rho_pF^3H^3 + (\rho_p - \rho_a)(H + 2FH)\left\{ \frac{\rho_pF^2H^2 - \rho_a(H + FH)^2}{(\rho_p - \rho_a)(H + 2FH)} \right\}^2 = 0 \]

Simplifying,

\[ \rho_a(H + FH)^3 - \rho_pF^3H^3 + \frac{\rho_pF^2H^2 - \rho_a(H + FH)^2}{(\rho_p - \rho_a)(H + 2FH)} = 0 \]

Extending,

\[ \rho_aH^3(1 + 3F + 3F^2 + F^3) - \rho_pH^3F^3 + \frac{\rho_pH^2F^2 - \rho_aH^2(1 + 2F + F^2)}{(\rho_p - \rho_a)H(1 + 2F)} = 0 \]

Multiplying across and dividing by \( H^4 \),

\[ \rho_a(\rho_p - \rho_a)(1 + 5F + 9F^2 + 7F^3 + 2F^4) - \rho_p(\rho_p - \rho_a)(F^3 + 2F^4) + \rho_p^2 - 2\rho_p\rho_a(2F^3 + F^4) + \rho_a^2(1 + 4F + 6F^2 + 4F^3 + F^4) = 0 \]  

(9b)

Substituting the values of \( \rho_p \) and \( \rho_a \) gives a simple expression in \( F^4 \), from which \( F \) is obtained and therefore \( D \).

**Example.**

If \( \rho_a = 30 \) and \( \rho_p = 400 \),

then

\[ \rho_a(\rho_p - \rho_a) = 111,100 \]

\[ \rho_p(\rho_p - \rho_a) = 148,000 \]

\[ \rho_a^2 = 900 \]

\[ \rho_p^2 = 160,000 \]

\[ 2\rho_p\rho_a = 24,000 \]

and formula (9b) becomes

\[ + 111,100 + 55,500F + 99,900F^2 + 77,700F^3 + 22,200F^4 = 0 \]

\[ - 148,000F^3 - 296,000F^4 \]

\[ + 160,000F^4 \]

\[ - 24,000F^2 - 48,000F^3 - 24,000F^4 \]

\[ + 900 + 3,600F + 5,400F^2 + 3,600F^3 + 900F^4 \]

\[ 12,000 + 59,100F + 81,300F^2 - 114,700F^3 - 136,900F^4 = 0 \]

Simplifying,

\[ F^4 + 0.837F^3 - 0.593F^2 - 0.432F - 0.088 = 0 \]  

(10)

Differentiating,

\[ 4F^3 + 2.51F^2 - 1.19F - 0.43 \]  

(10a)

For a first approximation try \( F = 1 \). Substituting in formula (10),

\[ 1.000 + 0.837 - 0.593 - 0.432 - 0.088 = 0.724 \]  

(10b)

and in formula (10a)

\[ 4.00 + 2.51 - 1.19 - 0.43 = 4.89 \]  

(10c)
DEEP FOUNDATIONS AND SHEET-PILING

The approximate correction of this trial value is obtained by dividing the remainder in formula (10b) by the remainder in formula (10c) and changing the sign. Thus

\[ F = \text{approximately } 1.00 - \left( \frac{0.724}{4.89} = 0.148 \right) = 0.852. \]

Again substituting in formula (10),

\[ 0.527 + 0.518 - 0.431 - 0.367 - 0.088 = +0.159, \]

and in formula (10a)

\[ 2.47 + 1.82 - 1.01 - 0.43 = 2.85. \]

Therefore \( F = \text{approximately } 0.852 - \left( \frac{0.159}{2.85} = 0.056 \right) = 0.796. \)

It will be seen by inspection that it is sufficiently accurate for practical purposes to assume that \( F = 0.79 \); if the process is repeated it will be found that \( F = 0.789. \)

---

**Fig. 53.—Penetration of Sheet-piles for Cantilevered Walls in Cohesionless Soils.**

Values of \( F \left( = \frac{D}{H} \right) \) have been calculated as in the foregoing for various values of \( \phi \) and, after multiplying the penetration by \( \sqrt{2} \), are given by the curves plotted in Fig. 53. The value of \( \rho_{p} \) used in the calculation allows for the effect of friction.

For comparison, curves A (obtained by Pennoyer (6.1)*) are given for the two cases of angle of friction of the soil against the sheeting of \( 0 \) and \( 0.6\phi \). Although not stated by Pennoyer it would appear that the Rankine passive resistance has been multiplied by two in obtaining these values of penetration. The broken curve B shows values of penetration frequently given in handbooks, ignoring friction.

* References thus (6.1) refer to the Bibliography on page 259.
Obviously the factor of safety against the bottom of a cantilevered wall from sliding forward is always greater than the factor of safety against tilting.

It frequently happens that the retained soil is stratified or partly saturated, so that the upper part of the pressure diagram, instead of being a single triangle, may be of some irregular form. In this case, take \( p_a \) as the unit increment of pressure just below the level of the ground in front of the sheet-piling and calculate the equivalent height \( H' \) such that \( p_a H' \) is equal to the pressure produced by the actual loading. Then, provided that the soil below lower ground level is uniform, the method of calculation already given may be adopted by substituting in (9) for \( p_a(H + D)^2 \) the expression \( 2 \left( P_1 + p_a H'D + \frac{p_a D^3}{2} \right) \), where \( P_1 \) is the total actual pressure above lower ground level, and making corresponding changes throughout, noting particularly the use of \( H' \) instead of \( H \) where appropriate. As \( P_1 \), and the corresponding moments about the bottom of the sheet-piling, must be calculated from the actual (not equivalent) loading, the usefulness of this substitution is limited to simplifying the formulae giving the intensity of pressure below the ground in front of the wall.

The case will now be considered of a single horizontal load applied at the top of the piling which projects above the level of the ground as in Fig. 54.

EQUATING HORIZONTAL FORCES, \( T + \frac{2(p_p - p_a)DX}{2} - \frac{(p_p - p_a)D^2}{2} = 0 \)

Therefore

\[
X = \frac{(p_p - p_a)D^2 - 2T}{2D(p_p - p_a)}
\]  

\( (11) \)
EQUATING MOMENTS OF THESE FORCES ABOUT THE BOTTOM OF THE SHEET-PILES,

\[ T(H + D) + \frac{(\phi_p - \phi_a)DX^2}{3} - \frac{(\phi_p - \phi_a)D^3}{6} = 0 \]

Substituting for \( X \), \( HT + DT + \frac{((\phi_p - \phi_a)D^2 - 2T)^2}{12D(\phi_p - \phi_a)} - \frac{(\phi_p - \phi_a)D^3}{6} = 0 \).

Extending,

\[ 12HTD(\phi_p - \phi_a) + 12D^2T(\phi_p - \phi_a) + D^4(\phi_p - \phi_a)^2 - 4D^2T(\phi_p - \phi_a) \]
\[ + 4T^2 - 2D^4(\phi_p - \phi_a)^2 = 0. \]

Simplifying \( D^4(\phi_p - \phi_a)^2 - 8D^2T(\phi_p - \phi_a) - 12DHT(\phi_p - \phi_a) - 4T^2 = 0. \)

Dividing by \( (\phi_p - \phi_a)^2 \),

\[ D^4 - \frac{8D^2T}{(\phi_p - \phi_a)} - \frac{12DHT}{(\phi_p - \phi_a)} - \frac{4T^2}{(\phi_p - \phi_a)^2} = 0 \quad \text{(IIa)} \]

Substituting values of \( \phi_p, \phi_a, H, \) and \( T \) gives a simple expression in \( D^4 \) from which \( D \) is obtained.

**Anchors for Sheet-Pile Walls.**

The provision of anchors tying back the top of a wall results in much more efficient design, since it permits a considerable reduction in the necessary penetration and strength of the piles. The anchorage of the ties for sheet-pile walls and bulkheads may consist of raking piles or anchor blocks, sometimes known as "deadmen," and examples are shown in Fig. 55. Raking piles are used

![Fig. 55.—Anchorages of Ties.](image)

where the soil cannot be depended upon to give adequate passive resistance to the sinking or sliding of anchor blocks or where circumstances do not allow anchor blocks to be placed outside the area immediately behind the wall which is within the probable plane of rupture. As shown in Fig. 55 tension is produced in one of each pair of raking piles, so it is necessary either to have sufficient pene-
tration of the piles to take this in friction or to add dead weight to the piles to cancel or reduce the tension. Two raking piles are more effective than one raking pile and one vertical pile, which would have half the resistance and would deflect twice as much in developing this resistance.

For ordinary sheet-pile bulkheads, anchor blocks are used in place of raking piles, principally because of the lower cost, although with anchor blocks the restraint to the ties (being dependent on the passive resistance of the soil near the surface) is not so definite.

The minimum distance behind the wall at which the passive resistance of the soil is assumed to be effective to restrain forward movement of the anchor block varies according to both the properties of the soil and practice as shown in Fig. 55.

American and British practice generally favours the method shown in Fig. 55 (a), although the method indicated in Fig. 55 (b) can be used when space is limited, but with loss of efficiency since the tendency is to increase the active pressure on the back of the wall. Where the soil penetrated is poor near the surface but good slightly lower down and the width available for new construction is restricted, method (c) is sometimes adopted. As no account can be taken of the soil in the area ABC and little of that in the area CBD the piles are subjected to bending, and this method should generally be avoided and pairs of raking piles used instead (Lower part, Fig. 55). The point B is generally taken as the tip or shoe of the sheet-piles, and sometimes, but incorrectly, at B’, where there is movement forward; it should be the point at which there is no movement which may be assumed to be at the bottom of the pile or about 4D above the bottom if the piles are fully fixed.

Since the passive resistance of the soil to the anchor block and the friction against sliding are both affected by changes in water content of the soil, and this is to be expected near the surface, the assumptions in design must be conservative, but where the opportunity occurs, without deep excavation, the ties can be inclined downward as in Fig. 56, with consequent shortening and greatly
increased passive resistance of the soil at the greater depth. Since movement of the anchor must occur before the passive resistance develops, turnbuckles on the ties are usually tightened after the load is on them. However, the passive resistance is variable with movement in the same way as active pressure, so care has to be taken against unduly increasing the stress in the ties without advantage to other parts of the structure.

Anchor blocks are invariably of concrete cast in place and are designed for three requirements:

(a) The block must not settle in the soil. This requirement is easily met in undisturbed soils, but where the anchorage must be in unconsolidated fill, piles would need to be used as Fig. 55 (c).

(b) The block must not slide forward. This may be resisted principally by either friction on a horizontal plane or the passive resistance of the soil to horizontal pressure according to which type of anchor block is used.

(c) The block must be designed for the moments and shears due to earth pressures transmitted by the tie-bar.

None of these involves considerations beyond those already mentioned, provided it is remembered that the friction to sliding forward, although it may involve two surfaces of contact, should allow for the reduced friction of wet soil.

For simplicity, the resistance of the anchor blocks may form a continuous beam behind the sheet-piling, and in some cases such a beam may be necessary.

In the more usual case of a separate block to each tie-bar, however, the passive resistance of the soil is greater because the wedge of soil fans out in plan. From experiments, the wedge of soil forced out by anchor blocks near the surface has the shape shown in Fig. 57 for clean sand, the angle \( \theta \) being consistently close to the angle of repose, instead of \( \left( 45^\circ - \frac{\theta}{2} \right) \) as might be expected, due to friction on the face of the block and the tie in the tests being fixed at 0.4 of the height. Presumably this applies also to other granular soils since the results were practically the same for shingle.

When the block was close to the free surface the tests showed that the additional soil forced out in excess of the straight-sided wedge of width \( b \) is, as was to be expected, not related to the width \( b \) but only to \( d \), and the movement to develop the Rankine value of the passive resistance increased generally with
the ratio \( \frac{b}{d} \) and also with the wetness of the soil. It is to be noted that the tie-bars are best attached to the anchor blocks at 0.33 of the height only in the unusual case where the top of the block is at the surface. It is not necessary to deduct from the passive resistance the active pressure on the back of the anchor-age; it is an unnecessary refinement except perhaps where the soil is definitely cohesionless.

The coefficient \( M \) relating to the passive resistance (Fig. 49) should not be applied unless the weight of the block and any ground on it exceeds \( \tan^{-1} \mu \) times the passive pressure.

**Anchor Ties.**

Tie-bars are usually of round mild steel with threaded ends and divided into two lengths connected by a turnbuckle for adjustment. Often the ends are upset so that cutting the thread does not involve a reduction of the section; nowadays this is often done by butt-welding together the two diameters of bar. The most efficient spacing of the ties is a compromise between the necessary size of the capping beam, or waling, stressed in bending and with maximum horizontal shear forces of half the tension in any one tie, and the increasing total cost of the anchor blocks and ties as the spacing is reduced. Generally the best spacing is that which does not lead to overstressing in shear, or bending, the minimum section of capping beam or waling which is necessary in any case to protect the top of the wall and to provide a fixing for the end of the tie-bar.

Steel tie-bars are tarred, wrapped with hessian and again tarred as a protection against corrosion, and if the bars are long and the soil is newly-placed fill, or of equally poor supporting value, they are sometimes supported intermittently by light vertical piles.

The cost of high-tensile tie-bars having a tensile strength of about 70 tons per square inch is about twice that of mild steel of 28 to 33 tons per square inch tensile strength, but the 0.2 per cent. proof-stress and the safe working stress is three-and-a-half times as great; it may therefore be economical to use high-tensile bars for anchor ties if the forces are great.

Rolled threads enable the full strength of the bar to be developed at the anchorages which is not normally possible with ordinary threads and nuts, which may necessitate either allowing for the reduced section at the threads or the provision of upset ends. However, if the bars are tarred and wrapped as is usual, allowance needs to be made for the greater extension of high-tensile tie-bars according to the working stress adopted. Alternatively the tie-bars can be prestressed to the full initial stress of 45 tons per square inch against an encircling member of concrete which combines protection and strut action and reduces the change of length under variation of load to that arising from the change of compression in the concrete, say from 1500 lb. per square inch to, say, 300 lb. per square inch. However, to allow for possible settlement of the filling, appreciable residual compressive stress in the concrete is often desirable. As an example, a concrete member 10 in. square may be prestressed centrally by a \( \frac{1}{8} \)-in. diameter high-tensile bar to a stress of 1000 lb. per square inch corresponding to a stress of 45 tons per square inch; the external load should then not exceed about 30 tons, with 40 tons as the limit at which there will be no compression in the
concrete when allowance is made for shrinkage and creep of the concrete. The extension under load of a 50-ft. tie-bar will be about \( \frac{1}{3} \) in. for a prestressed member, 2 in. for a high-tensile bar not encased or prestressed, or \( \frac{1}{2} \) in. for mild-steel tie. It will therefore often be found that high-tensile steel tie-bars are more economical, particularly if the spacing is not greater, or not much greater, than that of alternative mild-steel ties with walings of about the same size.

Reference should be made to pages 93 to 95 for particulars of the extra tension to be taken into account to allow for arching of soil.

**Friction on Driven Piles.**

With pairs of raking piles acting as anchors it is necessary to determine carefully the ability of the tension pile to resist withdrawal, and since data on the resistance of piles to extraction is not so readily available, the following may be found helpful.

A formula used on the Continent for granular soils is that of Dörr,\(^{6,2}\) which for rectangular piles with parallel sides gives the final resistance to penetration \( (R) \) to be

\[
R = wbdl \tan^2 \phi \left( 45 + \frac{1}{2} \phi \right) + \mu \omega l^3(b + d)(1 + \tan^2 \phi)
\]

in which \( b \) and \( d \) are breadth and width of the pile, \( l \) is the length of the pile in the soil, \( w \) is the density of the soil, and \( \mu \) is the coefficient of friction (see page 73). For piles subject to downward load the whole expression is used and a factor of safety of \( 1 \frac{1}{2} \) to 2 is recommended. Since the first term gives the resistance of the end bearing, the second term alone gives the value of the side friction. To take this value of the friction resisting downward movement to be equal to the friction resisting upward movement is to assume that the compaction by the downward driving is equally resistant to an uplift force, and this appears to be so in the case of piles driven deeply where there is no risk of the surrounding soil being lifted. The resistance to uplift will invariably exceed the weight of the pile plus \( 2(b + d)\mu \left( \frac{I - \sin \phi}{I + \sin \phi} \right) \frac{wl^2}{2} \), especially in cohesive soils, and a graduated factor of safety of 2 at \( \phi = 15 \) deg. to \( 1.25 \) at \( \phi = 35 \) deg. should be sufficient, since if the soil is non-cohesive the compaction during driving will tend to change the term \( \left( \frac{I - \sin \phi}{I + \sin \phi} \right) \) to \( \left( \frac{I + \sin \phi}{I - \sin \phi} \right) \), and if it is cohesive cohesion will substantially increase the apparent value of \( \mu \) shortly after the pile is driven. Since when a pile starts moving upwards the resistance to extraction drops noticeably, it is desirable to make cautious estimates of the friction, especially when the friction is to be obtained partly from newly-deposited filling and partly from soil that was previously close to the surface.

Assuming, as suggested by Mr. J. Porter,\(^{6,3}\) that piles driven in soft soils tend to develop a friction constant

\[
f = \mu \omega \left( \frac{I - \sin \phi}{I + \sin \phi} \right),
\]

and that piles driven into granular soils that are not already so compacted that driving develops the full passive resistance necessary to lift the surface tend to
develop a friction constant

\[ f = \mu w \sqrt{\frac{\tan \phi}{\tan \phi - \sin \phi}} \]

then Mr. Porter’s formula, as adjusted by the writer, becomes

\[ F = (b + d) f l^2 \]  

(13)
in which the values of \( \phi \) and \( f \) in the following may be substituted.

\begin{align*}
\phi \text{ in degrees} & \quad 10 & 15 & 20 & 25 & 30 & 35 & 40 \\
f \text{ in tons per square foot} & \quad 0.005 & 0.0072 & 0.0085 & 0.0095 & 0.011 & 0.015 & 0.020
\end{align*}

The factor of safety suggested by the writer is 1.5 to 2.0 for steady and varying loads respectively.

For cohesive soils, the formula generally used is

\[ \text{Safe load} = \frac{2(b + d)\mu CA + bdCB}{N} \]  

(14)
in which \( C \) is the cohesive strength, which may usually be taken as the shearing strength (lb. per square inch); \( A \) is a factor which is 1.0 for driven piles, 0.85 for bored piles where a heavy ram is used to force the concrete against the soil, and 0.75 for bored piles with the concrete not heavily rammed; and \( B \) is a factor varying from 7 to 9. The factor of safety \( N \) is \( \frac{14}{3} \) to 2. Since the term representing the support obtained by the friction on the sides of the pile is much greater than the term representing the resistance on the bottom, it is important to note that a group of closely-spaced piles may need to be also considered for failure as a group. For this case, consider the smallest circle enclosing the group in plan; the circumference of this circle replaces the term \( 2(b + d) \) in formula (14).

Mr. Bullen \(^{6,4}\) has given average values of the skin friction on driven piles, and these data are referred to in Chapter II.

In pile-pulling tests in alluvial soil reported by Mr. Leslie Turner \(^{6,5}\) the friction constant obtained was 80 lb. per square foot per foot of depth for 13-ft. timber piles 6 in. square in section, from which for piles 30 ft. long a conservative average friction over the length would be about 600 lb. per square foot. In clays the resistance to extraction is often greater than would be obtained by any of the preceding formulæ when the value for \( \phi \) used is the angle of internal friction (see page 69) instead of the apparent angle of repose for the particular type of clay as is sometimes given in handbooks. Using the angle of internal friction it is necessary to add for the cohesion that will develop between the soil and the pile. That this reaches high values is known by cases where the tops are pulled off piles in attempting to extract them. With soils that have small cohesion this effect is not so great and more variable, but as an example, in one test of eight concrete piles driven 15 ft. and 20 ft. into mixed strata of sandy gravel with some clay, the average pull required to start extraction was 582 lb. per square foot of buried surface.\(^{6,6}\)

If the work is large enough to justify engaging a separate contractor for the anchor piles, bored piles with the concrete cast in place and, in suitable soils, having an enlarged base, may be provided. Such piles have considerably greater resistance to extraction, but the actual resistance can be determined only by test.
Penetration of Sheet-piles.

Anchored sheet-piles may be driven either to just sufficient penetration to balance the lateral forces, or to a penetration sufficient to give partial or full restraint at the bottom of the piling. In the former case, unless there is a slight increase of the penetration, the only factor of safety against the wall sliding forward will be that due to ignoring cohesion of the soil in front of the wall. Although Pennoyer rightly states that from the viewpoint of the most efficient use of the material the results of providing the minimum penetration or driving to full fixity and obtaining reduced moments on the wall are almost identical, in practice it is usually most convenient to extend the penetration beyond the minimum depth until the bending moment on each pile is just less than the safe moment of resistance of the pile. At the same time it will be appreciated that the factor of safety against sliding forward is greatly increased by increased penetration, in fact, roughly as the square of the increase in penetration. The overall factor of safety of the wall is also increased, since if the properties of the soil are over-estimated, and simple support only is obtained instead of complete restraint, then the bending moments on the wall will be increased by only 40 to 50 per cent. and, as Pennoyer (6.1) has pointed out, will still be within the elastic limit.

The case will be considered first where the penetration is sufficient to provide fixity at the bottom of the piles. The pressures are assumed to be of the form shown in Fig. 58, in which \( h \) is the equivalent (not the actual) height above the level of the ground in front of the wall and that the point of contraflexure is at point \( O \) at which there is no pressure, the position of which is given by

\[
Y = \frac{\gamma a h}{(p_p - p_a)}
\]  

(15)
so that the extent of the piles above $O$ acts as a simple beam spanning between $T$ and $O$, and the tension $T$ and the horizontal reaction $R$ at $O$ are obtained directly. This assumption is only approximate; the true level is the level at which there is no shearing force, and the approximate and true levels should be compared upon completion of the calculations. The point of contraflexure is generally higher, and Dr. Blum \(^{6,7}\) has given the approximate depth below the bed of the waterway, if the soil is of uniform quality and the piles are driven deeply enough to ensure fixity; these depths are given in Table XI. The actual level of the point at which there is no bending moment approaches the bottom of the piles as the degree of restraint is reduced. The procedure, assuming full fixity, is as follows.

<table>
<thead>
<tr>
<th>$\phi$ (deg.)</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to zero moment</td>
<td>$0.25H$</td>
<td>$0.15H$</td>
<td>$0.08H$</td>
<td>$0.035H$</td>
<td>$0.007H$</td>
</tr>
</tbody>
</table>

Equating horizontal forces and multiplying by two,

$$2R + (2D + h)(\dot{p}_p - \dot{p}_a)X - [(\dot{p}_p - \dot{p}_a)D - \dot{p}_a h]Z = 0$$

$$X = \frac{[(\dot{p}_p - \dot{p}_a)D - \dot{p}_a h]Z - 2R}{(2D + h)(\dot{p}_p - \dot{p}_a)}$$

Equating moments of these values about the bottom of the piles, and multiplying by six,

$$6RZ + (2D + h)(\dot{p}_p - \dot{p}_a)X^2 - [(\dot{p}_p - \dot{p}_a)D - \dot{p}_a h]Z^2 = 0$$

Substituting for $X$,

$$6RZ + \frac{[[(\dot{p}_p - \dot{p}_a)D - \dot{p}_a h]Z - 2R]^2}{(2D + h)(\dot{p}_p - \dot{p}_a)} - [(\dot{p}_p - \dot{p}_a)D - \dot{p}_a h]Z^2 = 0.$$  

Substituting $Z = (D - Y)$ and multiplying across,

$$(6RD - 6RY)(2D + h)(\dot{p}_p - \dot{p}_a) + [(\dot{p}_p - \dot{p}_a)(D^2 - DY) - \dot{p}_a(Dh - hY) - 2R]^2 - [(\dot{p}_p - \dot{p}_a)^2D - \dot{p}_a(\dot{p}_p - \dot{p}_a)h](2D + H)(D^2 - 2DY + Y^2) = 0,$$

from which is obtained

$$D^4[-(\dot{p}_p - \dot{p}_a)] + D^3[(2Y - h)(\dot{p}_p - \dot{p}_a)] + D^2 \left[(2hY - Y^2)(\dot{p}_p - \dot{p}_a) + h^2\dot{p}_a + 8R + h^2 \frac{\dot{p}_p^2}{(\dot{p}_p - \dot{p}_a)} \right]$$

$$+ D \left[-hY^2(\dot{p}_p - \dot{p}_a) - 2h^2Y\dot{p}_a + 6Rh - 8RY - 2h^2Y \frac{\dot{p}_a^2}{(\dot{p}_p - \dot{p}_a)} + \frac{4Rh\dot{p}_a}{(\dot{p}_p - \dot{p}_a)} \right]$$

$$- 6RhY + h^2Y^2\dot{p}_a \left[1 + \frac{\dot{p}_a}{(\dot{p}_p - \dot{p}_a)} \right] - \frac{4RYh\dot{p}_a}{(\dot{p}_p - \dot{p}_a)} + \frac{4R^2}{(\dot{p}_p - \dot{p}_a)} = 0.$$  

\((16a)\)
For cases where a graphical method is not justified, and where the depth $Y$ to the point of contraflexure is known, the following formula, after Dr. Blum, may be used to obtain the penetration $D$ necessary to obtain fixity:

$$D = K \left( Y + \sqrt{\frac{6R}{P_a - P_p}} \right) \quad \quad \quad (17)$$

in which $K$ is a factor which is normally 1·1 but may rise to, but not exceed, 1·2 when the earth behind the lower part of the wall has a low angle of internal friction, and $P_a$ is in this case either the increment of the active pressure of the dry soil or the increment of active pressure of the combined soil and water, whichever is applicable, and usually the latter. As before, $P_a$ is Rankine's value multiplied by $M$ (see Fig. 49).

**Graphical Method of Design for Anchored Walls.**

The following examples of the graphical method are based on the assumptions given earlier, using Professor Jenkin's formula for the active earth pressure and Rankine's for the passive resistance. The method shown in Fig. 63 (pages 96 and 97) adopts the form of the pressure diagram given by Dr. Blum, but the penetration is the net penetration without any increase for a factor of safety.

It will be recognised, especially having regard to Jenkin's research, that for the passive resistance to be developed there must be movement and, since movement in front of the wall will be greater with the more flexible walling materials, there will be differences between the triangular distribution of passive resistance which is generally assumed and that actually developed by the soil. The latter will be a function of the modulus of soil elasticity and the properties of the wall. As the soil modulus is seldom known and is not constant for a given soil, the author doubts very much the usefulness of attempting at our present stage of knowledge to forecast the resulting variations of soil reaction, since in that case the cohesive properties of soils should be taken into account.

The usual assumption of a triangular pressure distribution of the passive resistance, ignoring the cohesive properties of the soil, is probably on the safe side if the soil is cohesive, but if cohesionless it is suggested that passive resistance be assumed not to commence until 1 ft. or 2 ft. below the surface of the bed of the waterway, as a combined rough allowance against erosion and over-stressing the soil near to the surface.

**Construction of Fig. 63 (pages 96 and 97).**

Calculate

$$h = \frac{\text{pressure at dredged level}}{P_a};$$

$$Y = \frac{\text{pressure at dredged level}}{(P_p - P_a)}.$$  

Construct the diagram of unit pressures down to point $O$. The principal points on the diagram are obtained as follows.

At the upper level of the ground, the weight (surcharge) on the ground is 4 ft. $\times$ 100 lb. = 400 lb. per square foot. $P_a = 400 \times 0.205 = 82$ lb.

At the level of the water in the ground behind the wall, the total weight of ground (or equivalent) is 400 + (11 $\times$ 100) = 1500 lb. $P_a = 1500 \times 0.205 = 307.5$ lb.

At the level where the nature of the ground behind the wall changes, the
effective weight of the soil immediately above this level is 100 - (0.6 \times 62.5) = 62.5 lb. per cubic foot. The effective total weight above this level is

\[ 1500 + (5 \times 62.5) = 1812.5 \text{ lb.} \]

Therefore \( P_a \) immediately above is \((1812.5 \times 0.205) + (62.5 \times 5) = 684 \text{ lb.}, \) and \( P_a \) immediately below is \((1812.5 \times 0.257) + 62.5 = 778 \text{ lb.} \)

At the level of the ground in front of the wall, the effective weight of the soil is \(120 - (0.7 \times 62.5) = 76.25 \text{ lb. per cubic foot}. \) The effective total weight above this level is \(1812.5 + (76.25 \times 6.5) = 2308 \text{ lb.} \) Therefore

\[ P_a = (2308 \times 0.257) + (62.5 \times 11.5) = 1312 \text{ lb.}, \] and

\[ \dot{p}_a = \frac{[76.25 \times 3.00 (Table VI) \times 1.78 (Fig. 49)] + 62.5 = 470 \text{ lb. per square foot}. \]

\[ \dot{p}_a = \frac{(76.25 \times 0.257) + 62.5 = 82.1 \text{ lb. per square foot}}{470 - 82.1} = 3.38 \text{ ft.} \]

\[ h = \frac{1312}{82.1} = 15.98 \text{ ft.} \]

Divide the diagram into any convenient number of strips, 1, 2, 3, etc., and calculate the total pressure corresponding to the area of each strip. Plot the total pressures along the base line of the moment vector diagram to any convenient scale and mark points 1, 2, 3, etc., corresponding to strips 1, 2, 3, etc. Join these points to the pole.

Project horizontal lines through the centre of gravity of each strip, and number the spaces between these lines 0, 1, 2, etc. In spaces 0, 1, 2, etc., draw lines parallel to lines pole-0, pole-1, pole-2, etc., in the vector diagram down to the point \( O \). The lines so drawn are part of the bending-moment curve. Produce the first and last of these lines to intersect; this intersection gives the centre of gravity of the pressures on the upper part of the wall.

By taking moments, determine \( T \) and \( R \).

Substitute \( h, Y, R, \dot{p}_a, \) and \( \dot{p}_a \) in formula (16a) and obtain \( D \). Calculate \( \dot{p}_a(h + D) - \dot{p}_aD \) and \( \dot{p}_aD - \dot{p}_a(h + D) \) and substitute in formula (16) to obtain \( X \). Complete the lower part of the pressure diagram and complete the vector diagram and the bending-moment curve as already described. The distance 0-40 on the vector diagram should correspond to the calculated value of \( T \).

Produce the line in space 0 to intersect the line of the tie-bar. A line joining this point to the foot of the bending-moment curve is the base line of the bending-moment diagram, and if correctly drawn should be tangential to the moment curve at the foot and should show no moment at the point of contraflexure. These conditions appear to apply to \( Fig. 63 \), but it will be appreciated that as the bottom of the funicular polygon is represented by straight lines, it is impossible to draw a tangent to it. If, as is usual, the point at which there is no moment is slightly above \( O \), then the sum of the active and passive pressures, the tension in tie, and the penetration, and hence the positive and negative bending moments, would all be reduced slightly.

In connection with the scales of the diagrams, the scale for the moment is given by

\[ (\text{Pole distance}) \times (\text{height scale}) \times (\text{scale of load on vector diagram}) = \text{in.} \times \frac{\text{in.}}{\text{in.}} \times \frac{\text{lb.}}{\text{in.}} = \frac{\text{lb.-in.}^2}{\text{in.}^2} = \text{in.-lb. per inch}. \]
Fig. 59.—Anchored Steel Sheet-pile Wall with minimum Penetration.
(See Fig. 60 for Vector Diagram for Bending Moment.)

Fig. 60.—Vector Diagram for Bending Moment.
(To be read in conjunction with Fig. 59.)
The scale for the deflection is given by
\[
(Pole \ distance) \times (height \ scale) \times (scale \ of \ areas \ on \ vector \ diagram) \times \frac{i}{EI} = \text{in.} \times \frac{\text{in.}}{\text{in.}} \times \frac{\text{lb.-in.}^2}{\text{in.}^2} \times \frac{\text{lb.}}{\text{in.}^4} = \text{in. per inch.}
\]

**Construction of Figs. 59 and 60.**—When it is required to reduce the penetration to a minimum, the graphical method in Fig. 59 is applicable. For this case, the total pressure on the front of the wall plus the tension in the ties must equal the total pressure on the back of the wall. There is no pressure on the back of the wall near the bottom if the penetration is only just sufficient to prevent the wall moving forward, and the problem is reduced to the following simple case.

Taking moments about \( T, Z \) is obtained directly from
\[
P_zg = \frac{(p_p - p_d)Z^2}{2} \left( t + Y + \frac{2Z}{3} \right) \quad \ldots \ldots \ldots \quad (r8)
\]
where \( P_z \) is the total active pressure above point \( O \), and \( g \) and \( t \) are as shown.

Generally, the penetration required is only one-half to two-thirds of that required for fixity, depending on the properties of the soil, but, as the overall factor of safety of the wall is not so great as with full penetration, conservative treatment in this case is of importance.

**Reduction of Bending Moment on Flexible Walls.**

The Danish rules (6.8), (6.9) of 1926 take account of the concentration of active pressure near the points of lateral support by assuming arching of the soil when the wall deflects. As the wall deflects, the slight expansion of the soil also reduces the total pressure, but as Professor Jenkin’s formula already recognises this the writer would not recommend for general use the combination of a reduction of the total pressure by the Danish rules and the use of Jenkin’s active pressure values.

In Fig. 61 the trapezium \( ADFO \) represents the active pressure, using any formula (say, Rankine’s) by which a hydrostatic pressure distribution results, and \( A \) is the point to which the tie is attached. The net pressure acting on the sheet wall below the tie is then the shaded area \( A'ADFOM'A' \), the curve \( OM'A' \) being a parabola with the horizontal axis such that
\[
MM' = q = \frac{k}{l} \left\{ \frac{10l'}{l} + 4 \right\} \frac{P_m}{l} \ldots \ldots \ldots \ldots \quad (r9)
\]
where \( P_m \) is the equivalent uniformly-distributed unit pressure on the wall between \( A \) and \( O \), that will give with simple supports at \( A \) and \( O \) the same bending moment as the load area \( ADFOM' \).

The pressure diagram \( ADFO \) is always very nearly a triangle; mathematically the moment at a level \( \frac{l}{4} \) (Fig. 61) is always \( \frac{1}{4}PL \) for a triangular, rectangular or any trapezoidal shape of loading diagram between these limits, and
\[
P_m = \frac{1}{4}(AD + FO).
\]
It therefore seems likely that the Danish rules intend the moment to be calculated at the point where it is greatest which, in the worst case of triangular loading, occurs at \( \frac{l}{\sqrt{3}} = 0.577l \) below \( A \); the moment is then \( \frac{2Pl}{9\sqrt{3}} = \frac{PL}{7.82} \), and the equivalent pressure \( p_m \) is \( \frac{1}{2}(AD + OF) \times (\frac{1}{6} \times 7.82) = 0.49(AD + OF) \).

Also, the expression in brackets in formula (19) only varies between 0.8 for \( l' = 0 \) and 0.86 for \( l' = \frac{l}{2} \); the latter case is seldom likely to occur.

---

**Fig. 61.—Danish Method of Modifying Pressure Diagram for Flexible Walls.**

The coefficient \( k \) is given by the empirical formula

\[
k = \frac{1}{I + \frac{0.01}{\sin \phi} \sqrt{\frac{(1 + n)Ea}{lf}}} \quad \ldots \ldots \ldots \ldots \quad (20)
\]

where \( \phi \) is, as before, the angle of internal friction of the soil; \( n \) is the ratio of the negative moment (if any) at the tie to the maximum positive moment (say, near to \( M \)); \( E \) is the modulus of elasticity of the material of the wall; \( a \) is the maximum thickness of the wall; and \( f \) is the permissible compressive stress in the material of which the piles are made.

For reinforced concrete piles \( k \) is generally between 0.7 and 0.85. For steel sheet-piles the combinations of other values of \( E, f \) and \( a \) often result in the same, or slightly greater, values for \( k \) as shown in Fig. 62. A simple means of obtaining the reduced moment is to apply the diagram in Fig. 64 (after Mr. R. N. Stroyer \(^6,10\)) which is based on a coefficient of \( \frac{2F}{I + F^{1.5}} \), in which \( F \) is the liquidity factor of the soil and is equal to \( \frac{1 - \sin \phi}{I + \sin \phi} \) for walls of a thickness of one-fiftieth of the span, and reducing as the wall becomes thicker and less yielding. The form of
this graph is substantiated by experience, but later work by Mr. Stroyer resulted in the coefficient being altered to \( \frac{2F}{I + F^2} \) for thin walls, but the writer is of the opinion that the tests on which the change was based justified more the continued use of the earlier expression. The existence of cohesion in the soil will probably reduce the moments even further, but no figures are available and those given can reasonably be assumed to apply.

\[ E = 3 \times 10^6 \text{ lbs/sq in.} \]
\[ f_c = 750 \text{ lbs/sq in.} \]
\[ n = 0 \]

\( \alpha \)

\[ E = 30 \times 10^6 \text{ lbs/sq in.} \]
\[ f_c = 15,000 \text{ lbs/sq in.} \]
\[ n = 0 \]

\( \alpha \)

(a) Reinforced Concrete Piles.  
(b) Steel Piles.

Fig. 62.—Coefficient \( k \) for Obtaining Moment-reduction Factor by Danish Method.

Using Fig. 64, and assuming that the cross-section of the pile selected has a thickness-to-span ratio of about 0.05, the reduction of the moments shown in the moment diagrams of Figs. 63 and 60 will be about 45 per cent., that is the sheet-pile would be designed for 55 per cent. of the full calculated moments shown, which ignore arching action of the soil.

Tschebotaroff \(^{10,11}\) made tests on large model flexible walls at Princeton and found agreement with the Danish rules in so far that the reduction in pressure due to arching occurred almost entirely below the level of the tie. However, the distribution of pressure for clean sand was very different from, and almost the contrary to, that assumed by the Danish rules. It was concluded that it appeared unadvisable to rely on arching effects in sand for a reduction of the bending moments, but rather the contrary if increased pressure by horizontal arching is possible. For clay and sand-clay mixtures, placing in layers and accelerating the consolidation by providing drains, was suggested. The lateral pressures would then be lower than the fluid pressures, but it does not appear possible to relate the pressures to the shearing strength of the soil as determined
Fig. 63.—Anchored Steel Sheet-pile Wall Driven to Fixity.

(See page 90.)
by laboratory tests and, particularly for cohesive soils, pressures obtained from empirical charts are suggested. In the tests referred to in the foregoing, pressures for cohesive soils were recorded above the critical depth, \( h_1 = \frac{2C}{w} \), contrary to expectation from Bell's formula, but in general the increase of pressure was less all the way down the wall, and at the bottom the pressure tended to be smaller than that calculated by Bell's formula.

The Code on "Earth Retaining Structures" \(^{6,12}\) recommends reducing the calculated moments by 25 per cent. for piles in cohesionless soil of uniform quality without clay or silt and not subjected to vibration and provided the deflection corresponding to the reduced moment is not less than 0.005\(l\). It is pointed out that any pressure from water in the ground is not affected. The Code also recommends that the tension in the tie should be calculated in the ordinary manner and then increased by 10 per cent. to allow for arching of the soil. For piles in cohesive soils, the Code recommends that the tension in the tie should be increased by 15 per cent. for this reason, although a reduction in the moment is not advised.

Brinch Hansen \(^{6,13}\) put forward the hypothesis that when an earth-retaining structure deflects the active pressure decreases on that part and increases on parts which become forced against the soil or which do not yield, so that failure will always occur in the manner assumed by the design. He developed a limit-design method on the assumption that the wall at failure develops plastic hinges and that the soil is in the condition of failure. Neither the flexibility of the
wall nor the compressibility of the soil enter directly into the calculations. Apart from failure of the tie-bar or its anchorage, or passive failure of the soil in front, failure will follow the development of yield-hinges in the wall according to the depth of penetration. Thus one yield-hinge may develop between the level of the tie and the dredged level; or two may develop including one in the embedded length, where there is enough penetration to develop partial or full fixity. P. W. Rowe,\(^{6,14}\) in reviewing this hypothesis, suggests allowing in the design of the ties for causes which increase the load, for example settlement of the backfill, and points out that the factor of safety against failure of the wall is not much greater than that against yield, since the moment does not decrease but continues to increase after the yield. He gives a graphical example of experimental determination of the most economical wall in which the negative moment on the wall at the tie equals the maximum positive moment below.

It seems generally agreed that bending moments decrease with the increase in the flexibility of the combination of the wall and the soil. Kerisel\(^{6,15}\) says design should be based on a factor of the first yield-stress of the wall. Rowe\(^{6,15}\) points out the deflected shape of the wall is a function only of the material of which the wall is made, that the deflection causes a large volume increase behind the wall and in the case of dense sand is accompanied by a reduction in the angle of internal friction with consequent substantial increase in the total active load. With loose sand the subsidence of the surface reduces any fixity above the tie level and increases the maximum moment. However, these points are to some extent secondary, and if the design procedures explained earlier are used, it is only desirable, in addition, to use conservative stresses in the anchors and the ties.

P. W. Rowe\(^{6,16}\) has also put forward a new approach to the design of cantilever walls and those anchored near the top. For cantilever walls in sand and with the same backfill close agreement appeared to exist with conventional design when friction on the wall is taken into account in the latter. For tied walls taking account of the flexibility of the wall and other relevant factors suggests that some economy may be possible by improved design. Since the saving of constructional materials obtained by designing piling for lower maximum moments can be a doubtful long-term economy, especially where corrosion may reduce the strength in time, the value of this research may be mainly for engineers to see what economy can be obtained without reducing the useful life or the present allowances for more adverse conditions than expected at the design stage.

**Design Charts.**

Design tables for sheet-pile walls would need to take account of so many variables that it is usual to make designs for each particular combination of soils, water levels, conditions of backfill drainage and level of tie-bars. However, with patience and assumptions, for example, that the backfill drainage is perfect and the tie-bars are one-eighth of the exposed height down the face, tables can be made. Two representative combinations are given in Fig. 65 for guidance when making trial designs. It is assumed in both cases that the water level at the front does not fall lower than the level of the channel bed. The soil taken for case A is coarse sand throughout. For case B the backfill is gravel, while the channel bed and sub-soil are fine sand.
Fig. 65.—Examples of Sheet-pile Walls with Ties for Two Typical Soil Conditions, showing Gross Bending Moment and Tension in the Tie (per foot of wall) and the Net and Recommended Penetration of the Sheet-piles assuming Perfect Drainage of the Backfill.
The penetration, the tension in the tie near the top of the wall, and the gross bending moment are all calculated on the same basis as previously described, using Rankine’s values for the active and passive pressures of the soil without taking into account the reduction of bending moment due to the flexibility of the wall, nor the increase of tension in the tie due to arching of the soil.

The drainage of the backfill can best be ensured, in the case of steel sheet-piling, by tidal flaps or by flame-cut weep holes, but with concrete sheet-piles the joints alone may be sufficient provided the wall is backed by hardcore or similar material. It is important, however, to allow for the additional moment and penetration necessary in cases where the drainage cannot be expected to be perfect. To do this it is necessary to ascertain the head of water behind the wall in the worst case to be expected and add for the full additional moment resulting, since no reduction of this additional hydrostatic moment is possible by arching of the soil.

Deep Sheet-pile Walls.

Where the exposed height of a sheet wall is greater than about 25 or 30 ft. it may prove economical to tie the wall at a lower level, particularly if this does not involve excavation for the tie-bar, thus reducing the moment in the wall,

![Diagram](image)

**Fig. 66.**

and the penetration, but increasing the size of anchor ties required. In this way, with a single level tie just above low-water level, the coping needs to be capable of preventing irregularity in deflection of the cantilevering sheet-piles. An alternative method to obtain this result consists of providing an additional tie-rod as in *Fig. 66 (a), (b), (c), and (d).* Of these the arrangement shown at (c) is often to be preferred, as with the others differing horizontal deflections at the two walings due to unequal extension of tie-rods and compression of soil in front of the anchorages are to be expected and necessitate provision in the design. The principles of design in cases (c) and (d) are the same as for walls with one tie.
In order to determine how the alternative design compares with that of a single tie at coping level it is sometimes convenient to calculate the moments and forces (as described earlier) for the normal wall, and the results may be used as a basis of trial calculations for the alternative construction as follows.

(1) Obtain the potential deflection $\delta$ of the single tie wall at the proposed level of the extra tie. Then the tension $T_2$ in this tie is that force which would cause an equal but opposite deflection in the single tie wall treated as a single span between the upper tie-rod and the centre of passive resistance [see Fig. 67 (a)]. It should be noted that if the filling is commenced before placing the lower tie-rod, causing an initial deflection $\delta'$, the force in the lower tie is reduced in the ratio $\frac{\delta - \delta'}{\delta}$, since the actual deflection $\delta'$ cannot be counteracted.

(2) The reduction of tension in the top tie-rod, and of pressure on the sub-soil, are respectively equal to the top and bottom reactions obtained by treating the wall as a simple span with a single point load equal to the tension in the lower (additional) tie-rod as calculated earlier. Allowance must be made for increased tension in both tie-rod if they are inclined.

(3) The penetration required with two ties will be less than that for one tie, so that the foregoing are approximations giving slightly high values for $T_1$ and $T_2$ and slightly low values for $P$. It is therefore advisable to multiply the revised minimum penetration required, calculated as described earlier, by a factor of safety of, say, 1.75 instead of $\sqrt{2}$.

The method described is approximate only, and is a quick way of trying the effect of different positions of the extra tie, but is not sufficiently correct for the final design of the sheet-piles using moment-reduction factors, the reason being that as the introduction of an extra tie reduces the ratio of length to thickness of the piles, the coefficients by which the gross calculated moments are multiplied will therefore often be greater than for a wall with one tie. There is also the complication that the ratio $\frac{P'}{P}$ in formula (19) is greater and tends to increase $q$.

The value of $(1 + n)$ in formula (20) is also greater and tends to reduce $q$. It is therefore preferable to omit the reduction factors if the wall has two tiers of ties.

The arrangements shown at (a) and (b) in Fig. 66 only differ from each other in the setting out of the anchor block (see page 101). The design calculations for both are identical, but differ from those for (c) and (d). This is due to the upper anchor block in (a) and (b) being so placed that in effect the whole of the anchorage force is transferred downwards by pressure through the filling to the lower tie which resists the whole of the tension, and the upper tie only serves to reduce the bending moment on the upper part of the sheet-piles which would otherwise act as simple cantilevers. The centre of gravity of the tension forces is thus much lower than in (c) and (d); hence the pressure to be resisted by the subsoil is reduced and there is a corresponding additional increase of tension in the lower tie. It will be seen from Fig. 67 (b) that the deflected form of the sheeting, and therefore the positions of the points of contraflexure, depend on the relation
DESIGN OF SHEET-PILE WALLS

between the upper and lower spans. The upper part of the piling and the upper tie-rod may be calculated with safety as if it were a normal single tied wall with a fixed-end at the level of the lower tie instead of as if simply supported by penetration into the subsoil, but no moment-reduction should be applied if the piles deflect inwards as in Fig. 67 (a).

The lower part of the piling is, however, quite indeterminate, as the exact pressure distribution, affected as it is by the transfer of pressure from above, cannot be determined with any accuracy. It may be assumed, however, that this pressure acts at the level of the lower tie, and on this basis approximate calculations can be made as follows.

1. Treat the lower part of the wall as a normal single-tied wall, with the tie at its top and ground level at the tie level, and with a superload equal to the actual superload plus the weight of the whole of the filling above.

\[\text{Fig. 67.}\]

2. From this, obtain in the usual way the value of the anchor tension, and add the tension in the upper tie-rod calculated as above.

3. Determine the bending moments and penetration of the sheeting in the usual way, neglecting moment-reduction, using the lower value of tension in the tie and neglecting the assistance from the continuity of the piling above, thus providing a small margin to allow for the possibility of a less favourable pressure distribution.

Owing to the allowances made to provide for the uncertain pressure distribution in cases (a) and (b) in Fig. 66, the final design may not always be economical compared with (c) and (d), but case (a) has considerably shorter tie-rods. This is a great advantage where construction space is limited, particularly where the filling is poor, and may also avoid the necessity of providing supporting piles under the tie-rods to prevent excessive sag.

The provision of an upper tie shorter than the lower is more typical of Continental practice, but this method was claimed by Ravier to have been shown, by model experiments, to be unsound. Although these tests showed the upper tie to take a larger part of a superimposed load than the lower tie the writer considers the tests were not conclusive. The method of Fig. 66 (c) is in the writer's opinion better than separate anchor blocks, especially when the soil is consolidated by loading or vibration.
Platform-type Walls.

A wall with a platform, of which typical cases are shown in Fig. 68 (a) and (b) and Fig. 69, takes longer to construct and is usually far more expensive than ordinary anchored sheet-piles, but for heavy superimposed loads, and also for non-tidal or very deep water, may form the only practical solution utilising sheet-piling. The advantages of this type are (1) Greatly reduced pressure on the sheeting; (2) Lower effective centre of pressure, and therefore (3) Lighter and shorter sheet-piling; (4) Greater resistance to impact and pull on bollards from large vessels berthing in deep water; (5) Greater supporting value of bearing piles, compared with sheet-piles, for the vertical loads; (6) Elimination of low-level tie-rods (a particular advantage in non-tidal waters); and (7) Greatly reduced construction width, especially with poor filling where the tie-rods would need to be very long.

The sheltered area under the platform could be left void, with openings to permit tidal water to enter and leave, but it is usually filled with light material such as clinker to prevent the slope flattening out after repeated rises and falls of the tide, thus increasing the pressure on the sheeting and also causing subsidence of the wharf surface behind the platform.

The pressure diagrams are as shown in Fig. 68 (c) and (d) respectively, and the moments and forces on the sheeting are obtained in the same way as for a normal anchored wall. The shaded areas represent the effective pressures and the dotted lines show the construction of the diagrams.
A variation of this system where the sheeting is at the rear of the platform is shown in Fig. 69. This has the effect of further shortening the sheeting and reducing the quantity of filling, but where it would be objectionable for the foot of the slope to project beyond the wharf face the saving is not very great and may be lost by the cost of hand-placed stone pitching, especially if placed under water.

A structure (6.17) with steel sheet-piles is illustrated in Fig. 70.
Sheet-pile Jetties.

Where a jetty projects into a flowing waterway, an open-piled construction is usual to avoid obstructing the free flow, and is also generally more economical than a jetty constructed of two lines of sheet-piles with filling between. However, if the jetty is to be formed by dredging on the outside of the two lines of sheet-piling and the original soil between already forms a substantial part of the earth core it needs only to have gravel, or if suitable the dredged material, added on top to make an economical construction. The two sheet-pile walls are best tied together rather than separately anchored.

![Diagram of Sheet-pile Jetty](image)

Fig. 71.—Sheet-pile Jetty.

Granular materials that are highly permeable are most suitable for the filling, for example, coarse sand or shingle, but if the use of less suitable materials, such as clay, cannot be avoided, special consideration must be given to the lateral pressure of the fill under the most unfavourable conditions of moisture content and frost. In this last respect the Canadian jetty (6.18) shown in Fig. 71, using a low level for the tie and separated anchors at the top of the walls, is a satisfactory type to withstand expansion forces due to frost, but in a moderate climate and with the same gravel fill it could have been tied straight across at the coping level.

Apart from the foregoing the design requirements are similar to single sheet-pile walls, and the conditions for permanent stability are better than for the equivalent double-wall cofferdams, since the forces from wave pressure will usually be negligible in comparison with those due to a substantial difference of water level.

Supporting Value of Sheet-piles and Resistance to Lateral Load.

Since sheet-piles seldom carry much vertical load it is not often that their supporting value needs to be calculated by the use of static or dynamic formulae, as in the case of bearing piles. As the penetration of sheet-piles is calculated to give adequate lateral support, the soil that will provide this invariably also provides ample support for such vertical load as there may be; for example, loads on the coping and the vertical component of the friction on the back of the wall. Methods of estimating the safe load on bearing piles are given in Chapter V. In the case of anchor piles which are driven vertically, the resistance of the pile to lateral load at the soil surface is in any case small, and in addition the movement to develop this is appreciable. In some tests of 30-ft.
round timber and concrete piles in river sand subject to lateral load, and tapering from 14 in. and 18 in. diameter respectively to about 10 in. diameter at the toe, the resistance to lateral load was 9 tons and 18 tons respectively for 1 in. of lateral movement and by proportion for smaller loads. Since the horizontal movement is greatly increased in soft soils, and at the same time the safe load on a given size of pile is reduced, it is most desirable to use raking piles as anchors.

Tests reported by L. B. Feagin {14,19} on timber piles driven up to 40 ft. in sand and with the tops of the piles set in large concrete blocks and subjected to lateral loads appear to be essentially a test of the passive resistance of the soil rather than the resistance of the piles. The following extracts show, however, the relatively large horizontal movement for moderate loads. Where \( \frac{1}{4} \)-in. horizontal movement is acceptable, \( 4\frac{1}{2} \) tons is suggested as the greatest sustained load on a pile; similarly where \( \frac{1}{2} \) in. is allowable, 7 tons can be applied if the load is steady. For alternating loads slightly lower maximum loads on each pile are suggested. The grading of the sand was such that all passed a No. 4 sieve and all was retained on a No. 100 sieve, but the proportion passing a No. 28 sieve varied from 8 per cent. to 56 per cent. These tests and pole-embedment tests suggest piles alone are only suitable for resisting small lateral forces. Inclined piles are better if resistance to extraction can be obtained or if they are loaded so that no tension can develop, or alternatively anchor blocks should be provided. If vertical piles are nevertheless desired, fins attached to the tops of the piles might be considered.

**Drainage.**

Too little consideration has been given to drainage, both in textbooks and in practice. The best form of drainage for a sheet-pile wall is by means of graded coarse material immediately behind the sheet-piling, and by weep holes also if the piling is steel sheeting or close jointed. Terzaghi has suggested that the shape of the grading curve of the filler material should be about the same as that of the filling and that the grains defined as being of the 15 per cent. size should exceed four times that of the filling, but be not more than four times that of the grains of the 85 per cent. size. (See also Code No. 2.) If this is not done, or is ineffective, there will be extra pressure behind the sheet-piling after rain-storms or at low tide. The extra pressure does not matter so much for the moments on the piles as for the effect the resulting underflow has on the passive resistance of the ground in front of the piles, which effect may be considerable.

Hence for sheet-piling in tidal waters either the drainage must be good or the factor of safety on the penetration must be greater; if neither apply, the wall will fail. In non-tidal waters drainage should be provided to prevent an excessive hydrostatic head following storms, or the top surface should be drained away, and stormwater thus prevented from temporarily building up behind the sheeting.

Should the reader doubt the importance of the preceding he may be convinced by applying the formulae on page 61 to any particular sheet-pile wall and determining the passive resistance of the soil after allowance has been made for loss of weight by the upward flow as well as buoyancy, noting also that the active pressure will be increased by the hydrostatic head causing the underflow.
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<tr>
<td>STEEL</td>
<td><img src="#" alt="Diagram" /></td>
<td><img src="#" alt="Diagram" /></td>
</tr>
</tbody>
</table>

**ADD FENDERING IF NAVIGABLE WATERWAY**

Fig. 72.—Types of Copings and Capping Beams.
Copings and Capping Beams.

Sheet-piling is usually provided with a capping, which serves as a stiffener to distribute lateral pressures evenly over the length of the wall. Lateral pressures may vary locally owing to the retained earth being unevenly consolidated, unevenly drained, or subject to forces from cranes, mooring of vessels, or local impacts such as from vessels.

For a simple cantilevered wall, the type of capping or coping used depends entirely on the local requirements, and is otherwise a matter for normal structural design. Where the wall is tied back by anchors the design of the capping beam must allow, in addition, for the moments caused by the horizontal component of the tension in the anchor ties.

The examples shown in Fig. 72 are typical of river walls and may need increase or modification of detail if used for wharves. For tidal waterways, if projections in front cannot be avoided, fender piles will be required, not so much to protect the structure from impact from vessels as to prevent small vessels from passing under and being held down on a rising tide.

Failures of Sheet-pile Walls.

A summary of the principal ways and causes of failure of sheet-pile walls is given in the following. These causes are additional to the obvious risks of over-estimating the properties of the soil and overloading the ground behind the wall.

Forward movement of the bottom of the wall may be due to insufficient penetration of the piles, dredging too deeply, or scour. Forward movement of the top of the wall may be the result of the anchorage being deficient or badly located, or because of excavation in front of the anchors, or ties being too small or corroded especially at the connections. Movement of the entire wall is probably due to a shearing failure of the ground.

Non-uniform filling or non-uniform consolidation of the filling behind the wall may result in the line of the coping becoming irregular. Subsidence of the filling may be due to any of the causes in the foregoing, or to the escape of the filling, especially if sand, between non-interlocking piles; or it may be due to the settlement of weak ground underlying the retained bank. Bulging of the sheet-piles may be caused by using an unsuitable material, such as mud, for the filling, or ineffective drainage of the filling; or it may result from weakening of the piles due to corrosion, rot or marine borers.

Omission to take into account arching action in the soil, and the consequent reduction of moments on the sheeting, and neglecting the reduction of active pressure due to forward movement of the wall, have no doubt resulted in many sheet walls being unduly strong.

Ample penetration and good drainage of the backfill are better investments than conservative design of the sheet-piles for bending stresses.

Note.—For Bibliographical References for Chapter VI see page 259.
CHAPTER VII

DESIGN OF COFFERDAMS AND METHODS OF DEWATERING

A COFFERDAM is a temporary structure to exclude water and to enable the construction in the dry of foundations, bridge piers, and the like, or a sheet-pile enclosure on land, in waterlogged earth, for the same purpose. The cofferdam method enables the permanent construction to be carried out in the open air, the alternatives being caissons, monoliths, or cylinders, the last possibly in conjunction with piles.

The essential difference between sheet-pile walls and cofferdams is the drainage of the backfill with the former, to avoid the greater penetration and anchorage otherwise necessary. Cofferdams, on the other hand, invariably hold back the maximum hydrostatic head possible and consequently need greater support. Where open caissons can be used, these are often more economical for foundations of small plan area, but sometimes the advantage of the cofferdam method over caissons is the avoidance of compressed-air work.

Subsequent examples will show that too many site factors are involved to make it possible to define simply the limits for the relative usefulness of the cofferdam and the various alternatives. Generally speaking, however, the cofferdam method is more likely to be the more economical the greater the plan area of the work to be constructed, provided the depth of water is limited to, say, 30 ft. or, more exceptionally, to 60 ft. The maximum is generally limited by cost, but, as sheet-piling is difficult to handle in lengths of over 60 ft., depths of water exceeding 40 ft. with soft soil, or about 50 ft. with rock bottom, become special problems.

The deepest cofferdams so far constructed are probably those of the Grand Coulee dam (90 ft.) and the Kensico reservoir (85 ft.), both in the United States, but these involve a method, described later, which takes them outside the preceding generalisations.

Types of Cofferdams.

There are two general types of cofferdam. The first is a relatively small construction box-like in plan, in which the work to be carried out is done between the internal strutting; the second is the type where a much larger area is enclosed, as for large river works and dams, where the new work is carried out in the open with the cofferdam construction enclosing it.

The principal types and, in a general way, the scope of their use are shown in Fig. 73, but it will be appreciated that it is not possible to do more than indicate the conditions favourable and unfavourable for each type. Thus, for rivers, the choice depends not only on the type of river bed and the soil strata penetrated, but partly on the relative availability and cost of materials at the site, and often on other factors, which are discussed later, such as the velocity of flow and probability of scour. Nearly every site has particular problems of its own, but no general agreement exists on choice of type for given conditions.
### Single Wall Types

<table>
<thead>
<tr>
<th>Location</th>
<th>Type</th>
<th>Suitable For</th>
<th>Not Suitable For</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>On Land</td>
<td>Steel Sheet Pile</td>
<td>Average conditions, especially in sheets sealed in clay</td>
<td>Subsoil with boulders below found. Water level of sheet piling in sheet pile unable to form seal, or non-impermeable strata.</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Steel or Timber Pile &amp; Horizontal Sheeting</td>
<td>Ditto</td>
<td>Ditto</td>
<td>2</td>
</tr>
<tr>
<td>In Water</td>
<td>Sheet Pile</td>
<td>AVERAGE CONDITIONS, ESPECIALLY IN SHEETS SEALED IN CLAY</td>
<td>DEEP WATER UNLESS PILES INTO CLAY</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Sheet Pile with Butressing</td>
<td>Still &amp; Slow Moving Water, Where Clear Distance Working, Space Essential</td>
<td>Exposed Heights Sufficient to Permit the Interlocks, Also Poor Soil in Channel</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Crib</td>
<td>Rock Bottom &amp; Risk of Flooding</td>
<td>Most Other Soils</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Movable</td>
<td>Several Uses on Same Site</td>
<td>Waterways with Stagnant Currents</td>
<td>6</td>
</tr>
</tbody>
</table>

### Types for De-watering Large Areas

<table>
<thead>
<tr>
<th>Location</th>
<th>Type</th>
<th>Suitable For</th>
<th>Not Suitable For</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>On Land</td>
<td>Sheet Piling, Butressed</td>
<td>Soft bottom and where top surface not accessible for anchoring.</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sheet Piling Anchored</td>
<td>Normal Conditions.</td>
<td>8a &amp; 8b</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sheet Piling Butressed</td>
<td>Rock Bottom</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>Slow Flowing &amp; Still Waters</td>
<td>Cellular</td>
<td>Where space does not permit beam sufficient to reduce hydraulic gradient or seepage.</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cellular</td>
<td>Rock Bottom</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Earth</td>
<td>Shallow or Tidal Waters</td>
<td>Soft clay Sub-stratum</td>
<td>12</td>
</tr>
<tr>
<td>Still &amp; Flowing Water</td>
<td>Ohio Type (Shelf Water)</td>
<td>Rock Bottom, (Construction from Barges)</td>
<td>Soft bottom unless ample protection against erosion.</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>Crib</td>
<td>Rock Bottom and Swift Current</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Inclined Double Crib &amp; Staggered Cribs</td>
<td>Rock Bottom, Swift Current and Possibility of Flooding</td>
<td>15(a)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Double Sheet Piling</td>
<td>Normal Cases of Soft Bottom</td>
<td>Penetration of Piles Limited by Rock Bottom Close to Channel Bed</td>
<td>16</td>
</tr>
</tbody>
</table>

*These types are only suitable for "be with pile sections having strong interlocks.*

---

Fig. 73.—Principal Types of Cofferdams.
The cofferdam must of itself be stable as a whole; for example, a cofferdam subject to an unbalanced horizontal force due to an earth surcharge on one side will produce vertical forces in the sheet-piling which may affect its watertightness and stability. This must be avoided, for example, by reducing the surcharge on one side and moving it to the opposite side.

Soil having boulders will cause difficulties in driving sheet-piling, while with hard rock cofferdams have the disadvantage of an open joint where the sheeting meets the rock and, if it is below the original soil surface but not below the level of the subsequent excavation, leakage along this joint cannot readily be prevented.

It is necessary for the sheet-piling to penetrate the soil sufficiently to prevent undue ingress of water through the sub-soil, and it is necessary that the type of sheet-piling used shall itself be, for practical purposes, watertight, not only as it is driven but when it is subsequently deflected by the pressure of water or saturated earth on one side only. With type 14 (Fig. 73) the framework is lowered down in sections as a timber skeleton and the timber sheeting fixed subsequently. It is only suitable for cases where underflow will be negligible and for small depths of water.

Where cofferdams are used for protecting the construction of foundations in waterlogged soil on land, and provided the piles can be driven to sufficient penetration, the shape of the cofferdam in plan may be the smallest rectangular shape which encloses the new construction, thereby keeping to a minimum the peripheral length and the timber or steel required in strutting, or for more important work may be circular with the thrust resisted by walings in the form of rings and without transverse struts.

Some allowance for a berm and drainage channel around and clear of the new construction is necessary if the seepage is likely to be large.

With small cofferdams on land it is usual to withdraw the sheet-piling after completion of the new work, using one of the methods described on page 53.

Where the cofferdam is used in a tidal waterway or a river, the shape in plan will be decided by the necessity of reducing to a minimum the obstruction to free flow of the waterway, and particularly by the need to reduce scour of the river bed along the outside of the sheeting.

In waterways, steel sheet-piles are frequently cut off at the level of the top of the new concrete by flame, and only the top length removed, that remaining being a permanent protection against scour.

As steel sheeting is a comparatively expensive item, it is sometimes worth while to ensure that the lengths driven give, on subsequent cutting, lengths to be recovered long enough to be suitable for further use instead of becoming scrap.

An alternative to the single line of sheet piling (type 16 of Fig. 73) is a double line of sheet-piling with soil filling (Fig. 74), which may preferably be sand or other granular material so as to reduce the strength of sheet-piling required. The two lines of sheet-piling may be tied together at the top with walings on the outside of each line of sheet-piles, but if it is possible to place the ties at a lower level, say in tidal waters, economy will be possible in the sections of sheet-piles needed.

For stability of this type of cofferdam wall, two conditions need to be examined. In case (a) of Fig. 44, with no water on either side, the construction
resists only the bursting tendency of the sand or other fill. If this condition
follows a rapid lowering of the water level, allowances need to be made for the
super-saturation of the filling and the effect, previously mentioned, of reducing
the angle of internal friction. If a line drawn at this angle from the bottom
of the piling when the penetration is the minimum, or from the point of contra-
flexure when driven to fixity, intersects the upper surface, the condition is similar
to that of two walls back to back as described previously, except that the moment-
reduction coefficient cannot be applied with safety. If the walls are so close
together that the line intersects both walls, the pressures must be calculated as
for a deep bunker, or silo,(7.1)* and irrespective of the dimensions there may be
in addition an internal pressure due to the water, but this depends on the efficiency
of the drainage.

The more serious condition to consider is with water on one side only. In this
case, with the sheet-piling driven to simple support only, the back line of piling
supports an inclined compression through the soil fill, as in Fig. 74 (b), and a single

![Fig. 74.—Double Sheet-pile Wall Cofferdam.](image)

tie at the top will in this condition be very lightly stressed. With the piles driven
to restraint, the lateral loading of the line of sheet-piles against the water will be
partly resisted by this piling as a cantilever, and the remainder will be transferred
through the soil filling to the inner line of sheet-piles.

It will be evident from the foregoing that the inner line of sheet-piling will
in this event not serve much useful purpose above about one-third of its exposed
height, beyond being a means of retaining the soil filling. It is possible, however,
in cases where ties can be fixed at lower levels, to utilise more fully the inner line
of sheet-piling as shown at Fig. 74 (c), resulting in economy of sections and in-
creased stability.

It should be noted, as with other similar types of cofferdam, that, unless the
sheet-piling penetrates into impermeable soil, under-seepage may reduce the
ability of the inner line of sheet-piles to take the downward reaction of the inside
wall, and as recommended later a berm on the inside may become essential to
obtain proper stability. Since filling placed between rows of sheet-piles is not
generally compacted and may settle, tie-bars should be either free to incline at
the connection to the piling or enclosed in a fibre or metal tube of large diameter.
Filling comprising clean granular soil placed hydraulically is preferable as sub-
sequent settlement is reduced and a smaller pressure is exerted against the piling
because of the larger angle of internal friction.

* References thus (7.1) refer to Bibliography on page 137.
Watertightness.

Water will enter the cofferdam in two ways: (a) by leakage through the sheet-piling, and (b) by underflow as indicated in Fig. 75. Practically all types of steel sheeting provide a reasonably watertight wall by reason of a practically continuous contact line in the interlocks of the piles when the wall is deflected by the lateral loading. Percolation through the interlocks is reduced by causing fine material, say, a mixture of ashes and sawdust, to pass into the leak from the water side, or alternatively the interlocks may be greased before driving so that fine material carried by seepage may lodge and seal the gap. More serious leaks may be reduced by a tarpaulin secured over the area concerned while the repair is effected, and stiffened so that it is not forced through any gap.

Underflow.

To prevent excessive underflow necessitates penetration sufficient, taking into account the permeability of the soil, to prevent the water coming in faster than it can conveniently be pumped.

Where sheet-piles have to be driven hard to obtain sufficient penetration to prevent excessive underflow, the ends of the sheet-piles may become buckled and the interlock may be so damaged that the succeeding sheet will be forced out, with the result of considerable inflow of water being revealed when the interior is being dewatered. This can sometimes be only suspected before the excavation has proceeded to some depth, and in such cases a "blisters" of sheet-piling may be driven immediately outside and the space between filled with puddle clay.

With permanent piling, say along the bank of a river, penetration of the sheeting into clay or other impermeable strata is rather a disadvantage in preventing free drainage, and separate drainage should then be provided. In the case of cofferdams, however, to drive the sheeting through hard gravel to penetrate into an impermeable strata will be most desirable in order to reduce pumping.

Quicksand.—If the soil is permeable and the penetration insufficient, "boils" will occur in the interior of the dam and the inflow may make dewatering impossible.

Fine sands of approximately uniform size of grain are most susceptible to becoming quick. As stated by Terzaghi, for a spontaneous loss of strength, the sand must be very loosely packed and saturated, and the pore water must be unable to escape rapidly.

Glossop and Skempton (7.2) consider that the tendency for the grains to fall into a denser packing is fundamental. Eighteen sands which they found to be quick or unstable had about 60 per cent. of the grains between B.S. Sieves Nos. 200 and 72 (approximately 0·08 mm. to 0·20 mm.), that is, within the range of sizes applying to fine sand; however, uniformity is more decisive than the actual size, as shown by six examples with about 60 per cent. in the range 0·02 mm. to 0·06 mm. (very fine sand or silt).

Calculation of Seepage of Underflow.—With a knowledge of the type of soil between the bottom of the excavation and the bottom of the sheeting, and for a short distance lower, it is possible to estimate the seepage. The flow net method used here was first developed graphically by Forchheimer (7.3) and has been dealt with at greater length by Harza, (7.4) Casagrande, and others.
A typical illustration of seepage is shown in Fig. 75 and follows D’Arcy’s formula \( v = KS \), in which \( v \) is the average velocity of flow, \( K \) is the coefficient of permeability (Table XII), and \( S \) is the hydraulic gradient \( \frac{H}{L} \). \( H \) is the head causing flow, and \( L \) the curvilinear length in the soil in which the head is lost.

Fig. 75.—Seepage by Underflow.

It is important to note that water flows through a permeable soil in two different ways, according to the velocity, somewhat similar to the flow of water in a pipe, whereby at a certain low velocity a critical phase is reached after which increased flow is accompanied by turbulence and greater loss of head. The formula
applies to the majority of soils in which the degree of permeability is not very high and the hydraulic gradient S does not exceed 1.0. It will not apply to very coarse granular soils which are highly permeable, nor in the condition when “boils” are tending to develop in the floor, in which case the flow of water is both rapid and turbulent.

In Fig. 75, flow lines are shown broken and equipotential lines chain dotted. A unit length of a straight wall will be considered, and the soil is assumed to be isotropic. The flow is laminar, or similar to that in a number of separate stream tubes, and the velocity increases as they get closer and narrower, as around the bottom of the sheeting. The spacing of the flow lines is related to the spacing of the equipotential lines by \( \frac{\delta s}{\delta n} = r \), and, as the flow is in the direction of maximum pressure drop, the intersections of the two are always at right-angles and the flow and equipotential lines are perpendicular to the surfaces and to the wall respectively.

At some depth in the soil the flow net may be interrupted by an impervious surface, or this can be assumed at some depth below which the flow will not have measurable effect on the total seepage. Since the whole head \( H \) is lost in \( N \) spaces, if \( \delta H \) is the head lost in each curvilinear section \( N \delta H = H \) and the hydraulic gradient in any section is \( S = \frac{\delta H}{\delta n} = \frac{H}{N \delta n} \). The flow is \( \delta Q = K \frac{\delta h}{\delta n} \delta s \) and by combining \( \delta Q = K \frac{H}{N} \), and for unit length of the straight wall this discharge will be multiplied by the number of stream tubes, or for \( m \) tubes:

\[
Q = \frac{m}{N} KH
\]

(21)

When, as frequently occurs, the soil is anisotropic and has three or more times greater permeability in the direction of the stratification, which may be assumed here as horizontal, it is necessary to adopt the method developed and used by A. F. Samsioe of Stockholm (1930) to ascertain the equivalent value for the two combined. Thus if \( K_h \) and \( K_v \) are the respective horizontal and vertical coefficients of permeability,

\[
K' = \sqrt{K_v K_h}
\]

(22)

and it is necessary, in addition, to multiply the horizontal scale by a distortion factor which, assuming \( K_h \) is, as is usual, greater than \( K_v \), is

\[
q = \sqrt{\frac{K_v}{K_h}}
\]

(23)

The flow net is then constructed with the structure drawn to the distorted scale and the soil treated as being isotropic.

In Fig. 75 (after Harza) the forces acting on particles of sand at various points in the flow path should be noted. Thus at the ingoing surface the hydraulic thrust \( g_s \) and \( w_{se} \) the weight of inundated soil after deducting buoyancy, both act downward and tend to consolidate the soil. At the outgoing surface the resultant \( R \) will be small; if it is negligible the surface will become “quick”, and as soon as \( S = 1.0 \) the sand grains float and boils develop.
DESIGN OF COFFERDAMS

The flotation gradient mentioned applies to sands and soils consisting essentially of quartz having a specific gravity of 2.70, but is greater for materials of greater specific gravity. Thus if $F_s$ is the flotation gradient, $\beta$ the porosity of the soil which is the ratio of voids to the total volume, and $G$ the specific gravity,

$$F_s = (1 - \beta) (G - 1) \quad \ldots \quad (24)$$

The values from Lane (7, 4) and Wing in Table XII show the wide range of $K$ for typical soils, and show that close determination by percolation tests on samples of the soil is essential before attempting to estimate probable seepage.

Table XII.—Coefficients of Permeability $K$ for Typical Soils.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Cubic cm. per second per cm.$^3$ for 1 cm. thickness and 1 cm. head*</th>
</tr>
</thead>
<tbody>
<tr>
<td>River sand</td>
<td>0.041 to 0.266</td>
</tr>
<tr>
<td>Dune sand</td>
<td>0.0185</td>
</tr>
<tr>
<td>Beach sand</td>
<td>0.0089 to 0.0216</td>
</tr>
<tr>
<td>Undisturbed soil, fine sand to gravel</td>
<td>0.0163 to 0.316</td>
</tr>
<tr>
<td>Clay</td>
<td>0.0000000023 to 0.000023</td>
</tr>
</tbody>
</table>

* For cubic metres per day per square metre for 1 cm. thickness and 1 cm. head, multiply the values given by 864. For gallons per minute per square foot for 1 cm. thickness and 1 cm. head, multiply the values given by 12.27 to give Imperial gallons or 14.73 to give U.S.A. gallons.

Example.—Assume that the sheeting is an impervious continuous straight barrier with dimensions as shown in Fig. 75, and that, due to stratification, $K_1 = 0.07$ and $K_2 = 0.20$. Then $K' = \sqrt{0.07 \times 0.20} = 0.116$ cu cm. per second per square centimetre, or $0.116 \times 12.27 = 1.42$ gall. per minute per square foot.

Now from Fig. 75, $\frac{m}{N} = \frac{6}{14} = 0.428$, and from equation (21) $Q = 1.42 \times 0.428 \times 20 = 12$ gallons per minute per foot length of the wall.

This example possibly does not give a good picture of the likely quantities of seepage water to be pumped for large dams. Thus for the Nag Hammadi barrage on the Nile (Figs. 116 and 117), where the whole area is sand and there is no rock throughout the site, the average total discharge of the pumps during the working season was about 1,000,000 gallons per hour.

The modification of D'Arcy's formula made by Hazen may be used for an approximate estimate of seepage, based upon the effective size of the sand instead of the coefficient of permeability. Thus, if $V'$ is the velocity of the water in the soil in metres per day, $C$ is a coefficient (usually 1000), $\theta$ is the temperature in degrees Fahrenheit, and $L$ is the length of the seepage path, then with the effective grain size $D_{10}$ in millimetres,

$$V' = \left[ C(D_{10}) \right]^2 \frac{(\theta + 10)H}{60L} \quad \ldots \quad (25)$$

The great effect of temperature on the viscosity, and thus on the velocity of flow, will be apparent.

The effective grain size is the diameter of the largest particle in the smallest ten per cent. by weight of a sample of the soil, for example the value of $D_{10}$ is 0.2 for the sample of sand at Trempeleau, Wisconsin, shown in Fig. 76.

The flow of water into a cofferdam can be estimated by using Hazen's formula (25) in combination with the flow-net diagram, each stream flow band being dealt
with separately. The effect of loss of weight by upward flow on the passive resistance of the sheet-piling is considerable in the case of cofferdams, and must

be allowed for in the design in cases where the sheeting does not enter impermeable strata.

The usefulness of any method of calculating seepage will necessarily depend to a very great extent upon accurate data being available, but in any case the seepage cannot be expected to be forecast within very close limits.
Hurst and Leliavsky Bey claim that a saving of time in obtaining flow-net diagrams is effected by the use of magnetic fields. The foundation and the sheet-piling are represented by sheet iron cut to the shape of the cross-section and magnetised. Equipotential lines are formed by iron filings gently vibrated. If this is done on photographic paper and exposed to daylight, a record is obtained from which the flow net can be drawn, say by the circle method.

Messrs. White and Prentis,\(^{(7,6)}\) who were largely responsible for what is probably the most extensive use of cofferdams in recent times, in connection with the upper Mississippi river improvement, have given the results of actual measurements of the flow of water under the cofferdam at Trempealeau, Wisconsin, including the effects of varying the flow by increased penetration of the sheet-piling. The result of these measurements is shown in Fig. 77, the grain size distribution curve for the sand being that given in Fig. 76.

If the under-seepage is through a soil which is only highly permeable in certain spots, for instance, because of boulders or debris disturbed and forced down by the sheet-piling, a method may be adopted which was successfully used at Plymouth.\(^{(7,7)}\) If the position of the resulting "blows" can be fairly closely determined on dewatering the interior, a hole is formed to below the level of the excavation by driving a steel section, or a rail, and after withdrawal a grout pipe is lowered into the hole. After washing out the pipe with water, and forming a cavity at the bottom, a \(1:1\) grout of cement and fine sand can be forced into the sub-soil around the boil sufficient to seal it completely. In the instance referred to, the grout was successfully fed through a 2-in. pipe by gravity, but this may be considered due to unusually favourable conditions.

**Pumping.**

The estimation of the seepage will determine the expected minimum pumping requirements after allowance has been made for any leakage expected through the sheeting itself. However, the pumping installation provided should also allow for pumping out the interior within a reasonable time in the event of any unusual accumulation of water, say as a result of increased seepage or breakdown of pumps. To minimise soil disturbance and the possibility of scour immediately inside the sheet-piling, it is preferable that the interior should be dewatered from points not too close to the sheeting.

The height of a cofferdam will be determined with a view to preventing the inflow of water by flooding or waves. Occasionally the top will have to be at a lower level from other considerations, such as access; or it may be higher in order to obtain, on subsequent cutting, lengths of sheeting suitable for re-use. If the top is lower than flood level, the pumping installation may need to be adequate for pumping out the whole interior filled with water within a sufficiently short time as not to lengthen the construction period unduly.

With permeable soil a berm about 10 ft. in width is preferably left immediately inside the sheeting, and the permanent construction will normally be arranged to leave this berm as a margin as shown in Fig. 78. An adequate berm reduces the hydraulic gradient by lengthening the average distance for seepage, which by also making the incoming flow more horizontal reduces the tendency for quicksand effects. If the flow of water tends to become excessive, loading the berm with coarse material will postpone the commencement of boils.
There are two accepted methods of pumping in the case of cofferdams on land, namely, to pump the seepage water from a sump on the inside of the cofferdam and to lower the water level outside by the well-point system of pumping.

For cofferdams in waterways, and where the normal system is used for land cofferdams, drainage channels are carried around, inside the berm if any, behind the sheet-piling, and led to a sump with a screened intake enclosing the pump suction pipes. The pumps are usually placed on a framework immediately above, either just inside the sheeting, or, in the case of shallow land cofferdams, immediately outside, if the suction lift does not exceed 15 ft. to 20 ft. The pump delivery may then be carried to any convenient discharge, and in the case of cofferdams in waterways there is a definite gain in efficiency if the discharge pipe turns down into the outside water to make a partial syphon.

Centrifugal pumps are mostly used and may be driven directly by steam or by electric motors, the latter method being by far the more preferable, even, in the case of large contracts, justifying the provision of a generating set if a supply of electricity is not available. Since the pumps generally have a high efficiency over a limited range of output, and to avoid the consequences of breakdown of a single pump of large capacity, it is usual to have several pumps and use two or more at a time according to requirements. If they are all of the same size and pattern, this enables prompt repairs by keeping spare impellers and other parts subject to abrasion from fine sand which penetrates through the filters.

The suction pipes should be separate for each pump, and preferably provided with flanged joints, the suction end piece for a length of, say, 10 ft. being of armoured hose and fitted with a screened intake and a non-return valve. A gin wheel and fall rope should be provided for easy lifting out of the suction end from the sump. Also, to keep the suction pipe airtight, the flanges should be provided with rubber or composition gaskets; the same applies to the delivery if it is arranged to syphon, as there will normally be a slight vacuum in the top part of the delivery.

It is a convenience in starting the pumps if an open tank is connected to the
pump delivery through a strong, properly closing, valve to enable easy priming of the pumps if the foot valve should not be closing effectively. Efficient screening of the pump intake will reduce considerably the wear on the pumps. In the case of very small contracts, and where the quantity of water to be handled is small and the lift does not exceed about 15 ft., portable petrol-driven diaphragm pumps are frequently used, but as the discharge is open their usefulness is limited to cases where the pumps can be mounted at the level of the top of the sheet-piling. Pumps of this type have capacities of 3000 to 6000 gallons per hour respectively for 3-in. and 4-in. suction pipes.

**Well-point System of Dewatering.**

An advantage of the well-point system of dewatering for land cofferdams is the ability to lower the water table in water-logged ground before excavation is begun, so that besides reducing the lateral pressure on the sheeting, and thereby facilitating the fixing of walings as the excavation proceeds, it also minimises the likelihood and the effect of a sudden flow of fine material through any temporary gaps in the sheeting. The method can only be used in soils having a sand content exceeding 20 per cent. It is ideal for operations in running sand and can be effectively used in nearly all gravel soils.

A typical arrangement of well-point equipment is shown in Fig. 79. Formerly the riser pipe had a valve and screen for suction and was driven down, but a more recent development comprises a combined jetting-suction head as in Fig. 80. When water at a pressure of 50 lb. to 150 lb. per square inch (12,000 to 20,000 gallons per hour), according to conditions, is forced down the riser pipe A, water pressure on flange B, which is attached to jetting tube C, forces the tube down, compressing phosphor-bronze spring D until the bottom of tube C is in contact with the seat E of jetting shoe F. When the jetting water is turned off, spring D returns the tube C to its original position and rubber-covered wooden ball G floats up on to its seat H. Resistance to damage of strainer gauze and cage J is provided by the strong steel perforated strainer K. Free flow of water under suction is ensured because tube C rises 1½ in. from its seat in the shoe and there is an unobstructed flow into the bottom of the tube and up into the riser pipe.

The suction for each jet-well under the most favourable circumstances with a 2-in. riser pipe is about 2000 gallons per hour for a suction lift of 25 ft. Accordingly a large-capacity jet pump is desirable for installation of the dewatering system. For example, a two-stage centrifugal pump, directly connected to a 32-b.h.p. diesel engine, is suitable for pressures up to 100 lb. per square inch, or a larger engine and a three-stage pump if a pressure up to 150 lb. per square inch is needed. The dewatering is done by a diesel-driven self-priming unchokeable pump of size according to the number of well-points and the permeability of the soil. For example, for up to ten well-points under average conditions a 2½-in. suction pump is common for a small installation, while for fifty to one hundred and fifty well-points pumps having a 6-in. to 12-in. diameter suction and delivery pipes may be needed. A lever device for subsequent extraction of the riser pipes is shown by Fig. 81.

When dewatering is required at a depth greater than about 18 ft., suction pumping is inefficient, and in this case a well of 20 in. diameter, or larger, is sunk to the full depth and a 14-in. diameter pipe, with gauze covering narrow
slits in the lowest few feet, is placed in the well and surrounded by coarse permeable material like shingle, with a puddled clay seal closing off the annular space from above. A submersible pressure pump is lowered into each of the 14-in. diameter pipes. An early application of this method in this country was to lower the level of artesian water under a dock at Southampton so as to reduce the hydrostatic uplift that would have made excavation of the dock dangerous.
The well-point method is not suitable for clay but has been used for draining silty soil, for example, all below No. 200 mesh, but in this case the space around the suction screen is filled with sand to form a preliminary filter. The screens of the well-points include one with 60 squares to the inch so that coarse sand which will not pass a screen of that size is used as a filter to stop the silt. The water discharged is normally quite clear. One advantage of the method is that bleeding of fine material out of the soil is avoided.

A formula quoted by Sir Henry Japp (7,8) applicable to either method in the case where it is ordinary ground-water that has to be lowered (and not artesian water) is

\[
Q = \frac{\pi K (H^2 - Z^2)}{\log_e R - \frac{I}{N} \log_e (X_1, X_2, X_3, \text{etc.})},
\]

where (see also Fig. 82)

- \(Q\) = Quantity of water pumped in cubic metres per second.
- \(K\) = Coefficient of porosity of the strata,
- \(H\) = Depth of water-bearing ground,
- \(S = (H - Z)\) Lowering of water at observation point,
- \(X\) = Distance from well to observation point, and
- \(R\) = Distance to end of cone of depression or range of lowering in metres.

![Fig. 82.—Well-point Method of Pumping.](image)

For small work the open-drainage channel method is that most used, since contractors generally have suitable pumps available and can add to the pumping installation at short notice if necessary.

**Effects of Pumping.**

In the case of land cofferdams consideration must be given to the effect on any surrounding structures caused by pumping from the sub-soil. Generally speaking, pumping should have no effect on surrounding property provided the seepage water is not carrying with it fine material. Sand or other fine material
will normally only be washed out from the sub-soil if the flow of water is concentrated in a few places. It is possible, of course, to be pumping clear water from the sump but for fair quantities of silt and fine sand to be entering the cofferdam through a few definite points of leakage. For this reason some engineers favour open horizontal sheeting retained by piles driven at intervals (Fig. 83) because this method ensures any seepage into the excavation being spread over so large an area of the wall face that little or no soil is disturbed. If the sheeting is driven tight, care should be taken to minimise the flow at any definite points of leakage. If sheet-piling is driven to form a practically watertight wall surface it should then also be driven deep enough to obtain a cut-off in impermeable strata such as clay. If this is not possible consideration

should be given to the open sheeting method, since it then becomes impossible for any water pressure to exist immediately behind the sheeting above the level to which the excavation has proceeded. In consequence this greatly reduces the hydrostatic pressure causing under-flow and therefore the tendency for the floor to become "quick".

**Consolidation by Freezing.**

The consolidation of wet soils by freezing, or by chemical processes, is an obvious alternative to pumping if the freezing occurs naturally, but silty soils and some clays expand upon freezing so allowance for this action is necessary, say by leaving temporary gaps initially in the bracing of open-timbered pits and cofferdams or taking equivalent precautions. Artificial freezing has been used in mining, but not very often in foundation construction. Fig. 84 shows grain-size classifications (after Skempton) within which freezing is the best method, but it is not limited to soil of that range of grain-size classification. Freezing may be chosen to avoid removal of sub-soil water where settlement of the surface would otherwise occur. An example of this was the sinking of a 14-ft. diameter shaft 123 ft. deep in New York (7.9) to reach rock for the driving of the shaft for a sewer tunnel nearly 200 ft. lower down in the rock. The cylinder of soil to be frozen was 36 ft. in diameter for which two 125-h.p. ammonia-compressors and twenty-one brine pipes set in a 26-ft. diameter circle were used; the brine return pipes were each 6-in. diameter with a 2-in. flow-pipe inside. The freezing of such a column of soil may take about seven weeks.

Removal of the water from saturated silt is usually accompanied by marked
shrinkage which is often followed by settlement of the surface. The necessity to avoid surface settlement by pumping led to the use of freezing the soil for the rebuilding of a railway tunnel in Montreal.\(^{(7.10)}\) The foundations of buildings on both sides do not go down to rock, and there was a stratum of silt between.

![Diagram](image)

**Fig. 84.—Limits of Grain-size Classification for Freezing and other Methods of Soil Stabilisation. (After Prof. Skempton.)**

**Chemical Consolidation.**

Consolidation of soils to give a temporary resistance to penetration by water can be done by various chemical methods, one of which is the injection of sodium silicate with a suitable hardener such as sodium bicarbonate. A comparatively new method of consolidating porous formations with a liquid of low viscosity and good penetrability is the injection of a polymer, which is hardened or set after a short time by the simultaneous injection of a setting agent. This action provides an impermeable consolidated mass. Some of these polymers have elastic properties and are not so readily susceptible to fracture by impact or shearing forces. They can penetrate where cement cannot.

**Electro-osmosis.**

Clay and saturated silt can be stabilised by electro-osmosis. The effect of a direct electric current passing through clay is to induce flow of the contained water and ions to the cathode, so reducing the water content and sometimes also altering the chemical composition; both actions tend to make the clay more stable, that is to increase the cohesive strength. This method was used for a bridge\(^{(7.11)}\) in Ontario to stabilise a bed of loose saturated silt extending up to 70 ft. and 130 ft. below the surface which was previously so sensitive to slips that the driving of piles for the bridge piers had proved practically impossible. Two pairs of anodes and the cathodes were used, both being deep and the latter being wells extending down to the rock (Fig. 85).

The method is suitable for use in combination with the well-point method,\(^{(7.12)}\) the well-points being the cathodes and may be 10-in. auger holes with say 4-in. diameter tubes surrounded by a sand filter. The anodes may be steel tubes.
Fig. 85.

Up to 90 volts and current up to 30 amps are reported to have been effective in Germany,\(^{7,13}\) but less voltage is required for saline soil-water and if the spacing of the anodes and cathodes is about 20 ft. As the drying action is mainly near the anodes, these are normally placed where dewatering is required. The spacing is dependent on the conductivity, electro-osmotic permeability, pore tension, and the water content of the soil.

**Effect of Frost on Soils.**

The effect of frost, apart from providing increased shearing strength, is mainly to increase the volume of a soil. However, with fine-grained soils with large capillary height, the increase in volume exceeds that due to the expansion
of the water previously present as capillary water is drawn up and ice lenses are formed. The soils most subject to frost action are those with more than 10 per cent. of silt, that is more than 10 per cent. of grains less than 0.05 mm., if well graded, but if of uniform size of grain, according to Casagrande, more than 10 per cent. of grains less than 0.02 mm. A thaw following a deep frost melts ice lenses before the soil lower down has thawed and can cause almost complete loss of strength of the soil.

Soils susceptible to frost in southern England have been found by the Building Research Station to come within the grain-size grading limits A and B shown by full lines in Fig. 86 but will be conservative for the more severe conditions in North America. For the latter reference can be made to Hogentogler (7, 14) and the Proceedings of the Engineering Institute of Canada. After Casagrande, (7, 15) the lower limiting curves for uniform soils and graded soils subject to frost heave are shown by the dotted lines C and D in Fig. 86.

To reduce frost effects, lowering the water-table by pumping is the best method; it should be lowered to more than the capillary height of the particular soil. This height will be over 6 ft. for silt, but less for permeable soils composed entirely of larger grains. Where the lateral thrust due to soil expansion is to be avoided, open trenches or trenches filled with a coarse-grained soil can be considered, while for sheet-pile walls back-filling with granular soil is desirable also for reasons explained earlier.

Prevention of Freezing of Water.

Where flowing water is involved and warming is not feasible, the method recently used for a bridge sub-structure at Huntsville, Ontario, (7, 16) may be of interest as it is reported to have permitted the use of floating construction equipment at temperatures down to — 30 deg. F. The method was similar to that tried in the past with qualified success of calming waves or swell by an upward flow of air. In this case a plastic pipe with holes along its length was laid on the river bed in up to 25 ft. depth of water, and supplied by a rotary compressor on the land. The success was believed due to the forcing upward of warmer water from below, so the method would be less effective in still water and is dependent on the air being sufficiently warmed by the compressor.

Cofferdams in Flowing Waterways.

There are some risks to be faced in the use of cofferdams in flowing waterways, which even engineers and contractors with wide experience have not been able to avoid successfully, with the result that whole cofferdams have occasionally been destroyed. Sometimes failures have been due to unforeseeable circumstances, but probably more often through insufficient appreciation of site conditions.

A danger arising when the cofferdam is over-topped, say during a flood, is the washing-out effect of the great volume of water which may pour over the top of the sheeting. With earth filling behind or between two lines of sheeting the flowing water may scour out a great volume of the fill, and the water falling into the interior may concentrate at certain points owing to variations in level of the top of the sheeting, or, where the top is level, along the upstream side. This may wash out the berm, exposing the sheeting to greatly increased moments.
DESIGN OF COFFERDAMS

It may be equally serious where there is internal strutting of timber, and some members, insecurely fixed, come adrift.

Probably of all the dangers to cofferdams the most serious is a reduction in stability due to scour on the outside owing to increased velocity of the water during times of flood. Cases have arisen where scour of over 30 ft. in depth of the river bed has occurred; the resulting effect on the stability of the sheet-piling is not difficult to imagine. Scour can be reduced by depositing an outside berm of shingle or rock, dependent upon the velocity of the water, i.e. the faster the current the coarser the material must be. Another method is to lay a mattress composed of a timber frame with fascines of willow wired to it, the whole being loaded down with rock.

The dangers from over-topping must be avoided by means for letting in water, preferably from the upstream side, in sufficient time to fill the cofferdam before any water comes over the top; withdrawing one or two sheet-piles is an

Fig. 87.—Sluice Gates in Steel Sheet-piles.
unsatisfactory method and may be most dangerous owing to the resulting scour. Sluice gates in the steel sheeting as in Fig. 87, together with chutes, should be used so that the incoming water will not cascade into the interior, but will be led down to the floor well into the inside. The time in minutes required to lower or raise the water level of a cofferdam by means of sluices is, assuming no leaks or under-seepage, given by

$$T = 0.414A \sqrt{H_1 - H_2},$$

where $A$ is the area in plan of the cofferdam, $A_s$ is the total area of the orifices and $H_1$ and $H_2$ are the upper and lower water levels respectively between which the water level is changed, all measurements being in feet.

**Internal Strutting.**

In the majority of cases cofferdams for the foundations of bridges and buildings will consist of a sheet-pile enclosure of slightly larger area in plan than the permanent construction, strutted internally with timber or steelwork. The principal function of cofferdam bracing is to hold the sheet-pile walls apart, and although the term bracing is generally used it will be appreciated that it is only in exceptional circumstances and with low water outside that sheet-pile walls have any tendency to move outwards. Normally the braces act as struts subject to transverse loading. The transverse loading is the self-weight of the braces and accidental blows in any transverse direction from crane skips. This condition of a strut subject to transverse loads is already well covered in textbooks, and an approximate method is suggested later for use in the design of bracing.

The walings are subject to bending stress in combination with axial compression when, as generally occurs, the walings also transmit the hydrostatic pressure from end to end or side to side of the cofferdam in conjunction with the interior strutting. In addition, the self-weight of the walings causes bending stresses in a vertical direction between the supporting posts and tie bolts. For this reason the wales should be of robust section and the breadth vertically the same, or nearly the same, as the depth horizontally. The cross-strutting then takes the horizontal forces from the walings, and where, as is usually the case, the width of the cofferdam exceeds about 15 ft., if timber struts are used they are supported from the floor or braced diagonally, the latter method being adopted where there is a soft bottom or to leave the bottom clear for excavation.

Manufacturers of steel sheet-piles give tables for the spacing of walings for equal loading from the water level downward according to the strength of the sheet-piling being used, and these are usually based upon the steel sheeting being simply supported at the walings. Disregarding the continuity leads, however, to the reactions as calculated by the approximate method also being slightly inaccurate. Fig. 88 shows the maximum spacing of walings, taking continuity into account and ignoring it. If timber bracing is used the spacing of the walings vertically is usually reduced owing to the large section required for the struts when the walings are spaced at the maximum to suit the steel sheeting, and Fig. 88 is principally of use in connection with structural steel internal strutting.

It should be noted that when the successive walings are fixed and strutted during the pumping out of the interior, higher stresses will occur temporarily
Fig. 88.—Calculated Maximum Spacing of Walings Vertically below High-water Level, based on the Strength of the Sheet-piling.

Fig. 89.—Unbalanced Hydrostatic Pressure on Sheet-piling as the Water Level is Lowered and the Bracing is being Fixed in Successive Stages.

in the steel sheeting and in the walings and struts at the level immediately above the strutting being fixed. The increases are greater in deep than in shallow water, since the difference in outside and inside water level causes a pressure difference through the whole height as shown at (b) in Fig. 89. For this reason,
unless the method of construction avoids causing these temporarily increased forces, it is generally undesirable to exceed normal design stresses in the design of the strutting and walings and a bending stress of 18,000 to 20,000 lb. per square inch for steel sheeting. It is preferable to lower assembled framing into the dam before commencing pumping. Structural steel is more easily arranged for this method, but with either steel or timber care is needed to obtain a good fit against the sheet piling and yet to avoid large spaces which require packings.

**Timber Bracing.**

With timber bracing the frames are seldom spaced nearer together than 8-ft. centres or farther apart than 12-ft. centres, as it is on the one hand necessary to have good access for removal of excavated material, and on the other, wider spacing involves excessively large walings and struts.

![Fig. 90.—Cofferdam Bracing with Hardwood Wedges.](image)

![Fig. 91.—Cofferdam Bracing with Screw Jacks.](image)

The design stresses need not allow for the timber being wet since the loads are reduced if the dam is flooded, but it is necessary to use low bearing stresses at the meeting of the struts and walings unless hardwood packings (Fig. 90) or metal plates with screw adjustment (Fig. 91) are used.

Where the bracing is below the ground level outside, the walings should be forced outward sufficiently to prevent slips or cavities on the outside of the sheeting, but not much or it may induce forces in the struts approaching the passive resistance of the soil.

**Stresses in Timber Bracing.**—As the forces result mostly from hydrostatic pressures and self weights, the actual stresses in the bracing can be determined fairly closely. The procedure in erection must be decided first, so that it is known whether the lowest frame at any given time will be subject to a temporary increase in horizontal load from the water pressure while the frame immediately below is being fixed. Erection of the bracing as the dam is being dewatered, of which a typical detail is given in Fig. 90, normally involves this temporary increase of stresses.

The method of framing and lowering the bracing before the dam is dewatered avoids consecutive increased forces in the walings and braces when the frames comprise the walings and struts for the whole depth, but by using periods of low
water outside it is usually more convenient to place only the top two or three levels in this way. An example of this method is shown in Fig. 92. The timber bracing is generally loosely suspended from the sheet-piling, but where the penetration of the sheet-piles is limited and the water level varies considerably it is desirable to effect some more positive attachment, since, if the sheet-piles tend to move outwards at time of low water, seepage into the cofferdam during subsequent high water may be increased by loosening of the soil along the face of the sheet-piles. Tie bolts and external walings, as seen in Fig. 90, overcome this.

To prevent the braces penetrating into the walings by excessive end bearing against the side grain of the latter, it is usual to limit the end bearing pressure on the walings. A limit of 500 lb. per square inch has been suggested, but this, in the writer's opinion, is already excessive unless hardwood packings are used, and it is doubtful if any experienced timberman, if left to his own judgment, would be likely to reach this stress.

The stresses used for the timber bracing may be based upon British Standard No. 940, which closely follows the corresponding United States Standard A.S.T.M. No. D245 and the Canadian (C.E.S.A.) Standard No. A.43. An extract from Part 2 of the British Standard is given in Table XIII which gives the permissible stresses for beams and struts under fully-protected conditions subject to certain conditions as regards density, grain, and imperfections.

The following stress factors are suggested to cover variations in exposure.

For fully-protected use . . . . . . . 1·00
For conditions in which timber is exposed to the weather and may be occasionally wetted and dried again. . . . 0·85
For damp conditions in which the timber remains damp and may be considered as being wet continuously . . . 0·70

For the exposure conditions usually applying to temporary cofferdams the writer recommends the use of the stress coefficient of 0·85, since although the lower coefficient of 0·70, applicable to wet conditions, might appear more correct, present practice in this work does not suggest that such limitation of the stresses is necessary.
It is not possible to quote briefly the grading classifications for the various qualities of timber of each species, and one of the specifications mentioned should be referred to. These gradings are related to the quality and density of the timber, and in particular the extent of defects, the limiting sizes of knot holes, shakes and wane edges, and the proportion of summer wood to spring wood. *Fig. 93* is based upon a stress of 1000 lb. per square inch, so that the safe load for any

| Table XIII.—Working Stresses on Timber for Continuously Dry Conditions. |
|-----------------------------|------------------|------------------|------------------|------------------|
|                             | Bending stress (lb. per sq. in.) | Shearing parallel to grain (lb. per sq. in.) | Compression perpendicular to grain (lb. per sq. in.) | Compressive stress (lb. per sq. in.) | Modulus of Elasticity (lb. per sq. in.) |
| Douglas fir (coast)         | Dense Close-grained | Dense           | 1750            | 105             | 380             | 1300            | 1040            | 1200            | 1,600,000       |
|                             |                  |                 | 1600            | 100             | 350             |                 |                 |                 |                 |
|                             |                  |                 | 1400            | 84              | 380             |                 |                 |                 |                 |
|                             |                  |                 | 1200            | 80              | 350             |                 |                 |                 |                 |
| Pitch pine                  | Dense longleaf or shortleaf | Do. Dense longleaf | 1750            | 105             | 380             | 1300            | 1040            | 1200            | 1,600,000       |
|                             |                  |                 | 1600            | 100             | 350             |                 |                 |                 |                 |
| Western larch and Western hemlock |                 |                 | 1300            | 85–90           | 300–325         | 1000            | 800             | 1,400,000       |
|                             |                  |                 | 1000            | 68–72           |                 |                 |                 |                 |
| Red pine and Ponderosa pine |                 |                 | 1200            | 85              | 300             | 900             | 720             | 1,300,000       |
|                             |                  |                 | 960             | 68              |                 |                 |                 |                 |
| Canadian spruce, Sitka spruce and Engelmann spruce | | | 1100 | 85 | 250 | 800 | 640 | 1,200,000 |
|                             |                  |                 | 800             | 68              |                 |                 |                 |                 |
| Eastern hemlock, Jack pine, and Lodgepole pine | | | 1100 | 75–80 | 275–300 | 800 | 640 | 1,100,000 |
|                             |                  |                 | 880             | 60–64           |                 |                 |                 |                 |
| Amabilis fir and Balsam fir |                 |                 | 1100            | 75              | 175             | 800             | 640             | 1,100,000       |
|                             |                  |                 | 800–880         | 60              |                 |                 |                 |                 |

given stress can be easily obtained pro rata. When using the graph for short spans it should be noted that the shear stress will need separately checking. The reactions are each equal to half the load, and the shear stress intensity is a maximum at the half depth and equal to 3/2 times the reaction divided by the cross-sectional area, from which the maximum distributed load on a beam or waling, ignoring continuity, is $\frac{3}{4}$ times the area multiplied by the safe shearing stress.

In actual design work it is worth while marking on the graph the maximum
load on the depth of section line, or lines, for the width and grade of timber being used.

To design a timber bracing for a cofferdam a simple method is as follows. First, knowing the exposed height and therefore the water pressure at each depth, obtain a trial arrangement of spacing of the walings as a proportion of the

**WIDTH OF WALING.**

<table>
<thead>
<tr>
<th>Safe Load in tons</th>
<th>Width of Waling</th>
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<tbody>
<tr>
<td>2</td>
<td>9</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
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<td>4</td>
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<tr>
<td>8</td>
<td>3</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
</tr>
</tbody>
</table>

**UNFORMLY DISTRIBUTED LOADS FOR EXTREME FIBRE STRESS OF 1000 LBS/FT².**

**EFFECTIVE SPAN IN FEET.**

<table>
<thead>
<tr>
<th>Safe Load in lbs, per inch width</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
</tr>
<tr>
<td>1500</td>
</tr>
<tr>
<td>1000</td>
</tr>
<tr>
<td>500</td>
</tr>
<tr>
<td>0</td>
</tr>
</tbody>
</table>

**Fig. 93.—Chart for the Design of Timber Walings.**

maximum span that the sheet-piling will withstand (Fig. 88). A suitable proportion of the maximum span given in Fig. 88, in the case of timber walings, may be 0.6. If the first trial results in the walings having either too much load for the sections of timber available, or the vertical spacing of the walings chosen results in them coming inconveniently close together lower down, a further trial may be necessary. In this way the horizontal load due to hydrostatic pressure is found for the second waling down and, provided that the walings lower down are spaced in the same proportions as in Fig. 88, they will be subjected to the same
loading and therefore be of the same section. The walings will be supported laterally by the struts across the dam at regular intervals, and if any of the walings are continuous the reactions and moments at these intermediate supports will be greater but the deflection of the walings less than if they were in short lengths. By dividing the plan of the interior to provide braces at equal distances along the walings to the sides and ends of the dam, a suitable section for the walings can be found fairly closely in the following manner.

Considering the side of the dam, the lateral pressure on the second and lower walings as shown in Fig. 94, is \( \frac{\omega D^2}{2(N - 0.85)} \) with full continuity and \( \frac{\omega D^2}{2(N - 0.73)} \) if freely supported, in which \( N \) is the total number of walings. Each waling will have axial compression because it also supports the ends of the walings across the ends of the cofferdam.

The bending stress in the waling due to self weight may usually be ignored in the case of square, or nearly square, section walings. Therefore the section is suitable if the axial stress added to the horizontal bending stress, obtained from Fig. 93, does not exceed the stress permissible in short columns reduced in accordance with the data given by curve \( T \) in Fig. 21, page 34.

For the transverse struts the accidental transverse impact load could perhaps reasonably be taken as 1000 lb., in comparison with which the bending stress by self weight is relatively insignificant and may be ignored.

There will generally be some end eccentricity in the axial load on the struts but, provided the ends are sawn dead square and the design and workmanship are good, this may normally be disregarded.

**Concrete Seal.**

After completing the excavation inside the cofferdam, concrete is usually placed which may itself be the foundation concrete for the work being constructed, or may be merely a sealing layer to enable the foundation to be constructed under comparatively dry and clean conditions.

In the latter event the concrete seal may be taken right up to the sheet-piling forming the wall of the cofferdam. If the sheeting is to be subsequently removed, it is best to prevent bond developing in some way or to drive each sheet pile an inch or so, while the concrete is hardening.

Where there is under-seepage the flow will sometimes cause difficulty in placing
the concrete. In such cases a remedy is to use relief pipes by means of which the water pressure immediately under the concrete is reduced until such time as the concrete seal is set and hardened. The well-point system is suited to this method, as a number of well-points, say one to every 30 sq. ft. of the floor area in the case of fine sand, may be embedded 3 ft. to 5 ft. below the underside of the concrete seal to be placed, the water level kept down for the necessary period, and the well-points subsequently disconnected and sealed off.

An alternative method with ordinary pumping is to lay open-jointed pipes in coarse material, and to have one or more sumps into which the foot-valves and strainers of the pumps are buried in rubble or hardcore. In this case success will depend upon the drainage channels not becoming blocked with fine material before the concrete seal has been placed and hardened. A graded filter may be needed. The concrete seal has to be placed under water if the upward flow is considerable, and generally the well-point system is the only suitable method in that case. The use of relief pipes will then enable a considerable reduction in the thickness of the concrete seal.\(^{7.17}\)

To maintain the concrete seal in position undisturbed while further load is added will usually necessitate allowing the cofferdam to fill, or filling it by pumping it for any reason the pumping through relief pipes is interrupted by a breakdown.

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CHAPTER VIII
CONSTRUCTION OF COFFERDAMS

As an example of the increased use of internal strutting of structural steelwork and the improved working space provided, in Fig. 95 is shown a combination of structural steel and reinforced concrete walings and struts used for the cofferdam for a pier of Chelsea Bridge.\(^\text{[8.1]}\) The inside dimensions were 106 ft. by 26 ft., and the sheet-piling (69 ft. long) extended well into blue clay. In the design it was assumed that hydrostatic pressure might be developed down into the blue clay, due to the flexibility of the piling and the possible movement between tides. It was found, however, that very little pressure developed below the level of the blue clay, and, by making a channel around the walls of the dam at the level of one of the reinforced concrete frames, the little water that came through the sheet-piling could be collected and a perfectly dry bottom was obtained for the foundation concrete. Holes were drilled through the steel piling to detect any tendency for the clay to squeeze through, and the fact that no sign of movement occurred was, considering the superimposed water load on the sub-soil, held to be a confirmation of Bell’s formula, if any were needed.

For the North State Street bridge at Chicago,\(^\text{[8.2]}\) where an underground railway tunnel was also built across the river immediately below the bridge, it was necessary to prevent the load of the new bridge piers from bearing upon the subway tubes, and accordingly steel strusses were provided to bridge the subway and were lowered into each cofferdam (Figs. 96 and 97). The two top stays of bracing were fabricated as steel strusses and dropped into place as a unit. After dewatering, the cofferdams were excavated to the bottom of the subway tubes and the subway sections constructed inside the dam from the ends of the sunken tube sections to the back wall of the cofferdam, the bridge piers constructed around the subway, and the remainder of the trench filled with sand and clay.

An example of steel sheet-pile cofferdams with timber strutting is illustrated in Fig. 98, which shows the foundations under construction for the New Jersey tower of the George Washington bridge.\(^\text{[8.3]}\)

Another example of steel sheet-pile cofferdam with timber internal bracing is shown in Figs. 99 and 100. In this case Larssen No. 2 sheet piles 40 ft. long were used to enclose an area of about 30 ft. by 65 ft., and the permanent construction was supported on box piles made from pairs of Larssen sheet-piles 50 ft. to 55 ft. long, which can be seen being driven in Fig. 100.

For the channel piers for the Lake Champlain bridge structural steel bracing was used (Fig. 101). The contractor’s plan \(^\text{[8.4]}\) for placing the bracing below mud line, which was accepted with slight modification, was as follows: \((a)\) After excavating approximately 12 ft. of mud from the cofferdam, place all the horizontal bracing in this space in the form of a nest, and wedge the upper horizontal frame in place against the sides. \((b)\) Proceed with the excavation and, when conditions permit, lower the remaining frames until the upper one is at the desired level and wedge it in place. Proceed in this manner until all the bracing is in

\* References thus \(^{[8.1]}\) refer to Bibliography on page 259.

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Fig. 96—Cofferdams for the North State Street Bridge, Chicago.
Fig. 97.—Cofferdam Illustrated in Fig. 96 under Construction.

Fig. 98—Cofferdam for a Pier of the New Jersey Tower, George Washington Bridge.
Fig. 99.—Steel Sheet-piled Cofferdam with Timber Bracing.

Fig. 100.—Driving Steel-sheet Box Piles inside Cofferdam illustrated in Fig. 99.
place, connecting the various sets of horizontal frames by tie-rods to act as suspenders and spacers during the process. This plan proved satisfactory, except that in one case the contractor excavated more than was planned before wedging one of the frames in place, and as a result the sheet-piling buckled just enough to make it impossible to lower the remaining frames into place without cutting them apart and re-assembling them. As this had been done in considerable depth of water, and required six divers for several weeks, it would have been less expensive, and the results perhaps equally satisfactory, had all the frames been assembled and bolted in place by divers. Except where the buckling occurred, the sheet-piling was easily pulled.

**Arched Cofferdams.**

When a bulkhead is required across a lock entrance, the existence of lateral supports at the sides makes a horizontal arch suitable. This method was used for the reconstruction of Whitehaven Harbour, the sheet-piling receiving lateral support from horizontal steel beams bent to circular arcs as shown in *Fig. 102*. The sheet-piling penetrated into sand for partial fixity, and the stresses in the sheeting are closely the same as if the arch ribs were walings. In a case like this the participation of the piling in circumferential forces (say, by compression
in the interlocks) should be disregarded because play in the interlocks will permit initial stressless yield and the arch ribs should be designed for the full hydrostatic pressure.

Circular Cofferdams.

Apart from the possibility of driving sheet-piles to form a circle in plan, as referred to earlier, where a watertight lining is not necessary because, for example, of very low water level or because of it being possible to lower the water level by pumping, circular cofferdams can be constructed on land by driving vertically steel beams, called soldiers, to support steel or concrete walings circular in plan and supported by the soldiers. Essentially this is a combination of the method used for Whitehaven Harbour (described above) in conjunction with open-timbered planking described elsewhere.

An example of this method is the circular cofferdam 101 ft. in diameter and 64 ft. deep for a sewage pumping station in Louisville.\(^{(8,6)}\) The soldiers were spaced at 7-ft. centres and reinforced concrete walings were spaced vertically according to the pressure, that is generally as explained on page 130. The walings were cast with the soil as a support when the excavation reached that level, the first being about 17 ft. below the surface. The walings were supported by steel bars welded to alternate soldiers. An additional waling was provided at one level because of a failure of some of the supports. The timber lagging was in lengths of 6 ft. 6 in. and spanned from one soldier to the next, and was increased in thickness as the excavation proceeded deeper; 3 in. was used for the top 20 ft., 4 in. from 20 ft. down to 30 ft., and 6 in. below 30 ft. The soldiers were extracted on completion of the work.
CONSTRUCTION OF COFFERDAMS

Cellular Cofferdams.

With steel piling having strong interlocks it is possible to use cofferdams of cellular construction in which the piles are stressed circumferentially. The advantage of this type of cofferdam is the ability to retain great depths of water, and most of the deepest cofferdams so far constructed have been of this type. Of the two methods (types 10 and 11) of constructing cellular cofferdams shown in Fig. 73, the type utilising complete circles with short connecting arcs, of which an example is shown in Fig. 103, is the better all-round construction.

![Cellular Cofferdam at Barnhart Island, St. Lawrence Power Project.](Image)

Although not so economical in material as the diaphragm type divided by straight cross-divisions, with the circular cells each compartment can be filled with soil immediately the last pile is driven, whereas with the straight diaphragm type the filling must be done in stages to avoid unduly stressing the separating walls. To avoid undue distortion, sometimes the sheet-piles for the cross-division are driven to a slight curvature in plan which becomes straightened out by the difference in the level of the filling, but both in practice and theory the diaphragm type has disadvantages compared with the circular cell type which more than offset the initial economy.

It should be emphasised here that only sheet-piling sections having high interlock strength can be used for either type of cellular construction, and because the majority of sheet-pile sections rolled in Europe are not intended for this use the following examples refer to American practice utilising sections with straight webs; for sections MP101 and MP102, the strength of the interlock in tension is 12,000 lb. per linear inch; for sections MP112 and MP113 it is 16,000 lb.
Deeply corrugated sections are generally not suitable for this type of use. Referring particularly to the circular cell type of cofferdam (Fig. 103), the piling is driven from a template, and all piles are usually pitched and interlocked together before the first is driven.

Although cellular cofferdams have advantages and are particularly favoured for use in deep water, they can only be used where the sub-soil is resistant to the heavy local vertical loading caused by the deposited filling. Being essentially a gravity type of construction they are more suitable for use in still or slowly-moving water where the bottom is rock, or at any rate where no scour is possible that would affect the stability.

![Diagram of Cellular Cofferdams]

Fig. 104.—Stability of Cellular Cofferdams.

The cells must be designed for (a) stability against over-turning, (b) sliding, and (c) circumferential tension in the piling. The most unfavourable conditions must be chosen in each case. Assuming a depth of water $H$, and ignoring the fill between the short arcs connecting the circles, the necessary diameter of the cell is related to the width $l$ of parallel filling required for stability. If the width $l$ is found in the usual manner the diameter of the cell of the same stability is given by

$$\frac{4l^2}{2.356}$$

The resultant $R$ (Fig. 104) must intersect the base within the middle quarter for "no uplift" on the water side. As shown in the figure the banking is ignored, but since the total friction $F$ is less than the horizontal force $P$ the banking is, in this case, required to provide the additional resistance to sliding and to provide also the excess needed as a factor of safety.

With regard to resistance to sliding, the penetration, if any, of the sheet-piles into rock should be ignored, and sufficient resistance ensured by the friction between the filling and the subsoil at the bottom of the piling. The coefficients of friction that apply will be those for wet materials, for instance in the un-
favourable circumstances of wet sand filling on fairly smooth rock the coefficient will be about 0.3, but will be increased for rougher rock surfaces and will also be higher for gravel filling. The coefficient of friction of wet clay, loam, and mud against fairly smooth rock will vary from about 0.1 to 0.3. These materials should be avoided as filling. Clay filling is not desirable because of the high resulting lateral pressure when wet and its susceptibility to volume changes. Although a wide variety of materials has been used as filling, including clay and earth, a mixture of fine sand and gravel best fulfils the requirements of low pressure on the sheet-piling and resistance to seepage.

Since the soil filling inside the cells will be affected by the submerged conditions when the filling is first placed, and subsequently there will be percolation through the filling, no allowance should be made for arching in calculating the lateral pressure against the sheet-piling. Arching, either in the way previously discussed for straight sheet-pile walls or due to similarity with the conditions applying in the design of bins, by which arching is taken into account in formulae such as those of Janssen and Airy, should be ignored, and the evidence shows higher lateral pressures than Rankine's formula would give, due no doubt to the "at rest" condition of the fill.

With hydraulically placed filling the pressures developed have been found to be much less than those due to the combined effect of the water and the soil under fluid conditions, as the fluid conditions seem to be localised to the immediate vicinity of the filling being placed. The extent to which the water level rises inside the cell must be fully allowed for in calculating the lateral pressure, and, as seepage usually results in a sloping water surface across the cell, some assumption is necessary of an equivalent horizontal water surface according to the permeability of the fill.

The hydraulic gradients, as shown in Fig. 104, indicate saturation lines for a soil permitting only slight seepage, say, silt; for soils of greater permeability the saturation lines will start much lower down, although the seepage is greater. In practice, the maximum tension in the interlocks may be some distance up from the bottom of the cell, and will usually not exceed three-quarters of the calculated maximum at the base, but there may be comparatively wide variations in stress in the interlock due to differences of pressure. The factor of safety to be used with the ultimate strength of the interlocks as given by manufacturers should preferably be not less than 2.5.

The tension in the interlock may be calculated as if the maximum is at the base, thus allowing for local concentrations of stress, and taking the "at rest" pressure for the fill, or say for clean sand or gravel, \( = 0.42wDH/2 \) where \( H \) is the height of the soil fill, \( D \) is the diameter of the cell, and \( w \) is the density of the soil, damp if adequately drained.

A trouble that has sometimes arisen with cellular cofferdams is due to the cell piling enclosing a natural deposit of mud or silt immediately overlying the rock or other hard stratum to which the piles are driven. If this is not grabbed out and replaced by more stable filling there will be reduced resistance to sliding.

Clay deposited on the water side of the cells will reduce seepage, but clay as a filling to the cells, although much used in the past, results in greater lateral pressures without corresponding advantages. On the side to be unwatered it is
necessary to ensure there being no hydrostatic uplift by providing weep holes in the sheet-piles on the unwatered side.

If suitable coarse granular material, quarry waste or rip-rap is available the size of the cells may be reduced below that necessary to be independently stable against the outside water pressure by depositing, before unwatering, a buttressing of such material, as indicated in Fig. 83. This method is more usual than that of depending on the cell alone for stability as, apart from a reduced quantity of sheet-piling it has very real advantages, by both lengthening and flattening the paths for seepage and also by partially balancing the external lateral forces on the cells.

If the natural deposit of the channel bed is fine material, unless the penetration of the cell piling is considerable a banking of coarse material may need to be deposited against the outside of the cells as protection against scour.

Fig. 105.—Failure of Diaphragm Type Cellular Cofferdam.

A berm on the inside is a necessity when cellular cofferdams are bearing on a sand bottom, in order to lengthen and flatten the seepage lines, and the berm should be loaded with slightly coarser material than natural soil. Otherwise the stability of the cells may be lost by the soil inside becoming quick. A failure of this type is seen in Fig. 105.

Cellular cofferdams were used for the cofferdam, which is probably of record size, constructed for the French Navy in 1946 to 1948 to form a dry dock at Brest for the French battleship “Jean Bart”. An aerial view of the completed cofferdam is shown in Fig. 106 and a view after dewatering in Fig. 107. The floating caissons, which were used to form the dock gate, were designed by M. Freyssinet in prestressed concrete and are shown, with a typical detail of the junction of the cells of the cofferdams, in Fig. 108. Unusual and novel features of the design of the cofferdam include the use of steel sheet-piles of American section (Bethlehem S.P.6) in conjunction with clutches of a European type to connect the Senelle triangular junction sections. Also the cofferdam was
Fig. 106.—Cellular Cofferdams at Brest.

(This illustration and Figs. 107 and 108 are reproduced by courtesy of M. André Coïard, consulting engineer, and Campenon Bernard S.A., contractors.)
constructed with interlocking steps to avoid long lengths of piling which would be difficult and dangerous to handle. So that the sheet-piles would stand up during driving, a layer of rubble was deposited beforehand through which they were driven. The writer is advised that the cellular cofferdam proved sufficiently rigid for the gate opening to form, in conjunction with the superimposed reinforced concrete beam and side walls, an adequately watertight joint when the floating caissons were set in position.

![Fig. 107.](image)

The usual method of filling cellular sheet-pile bulkheads by pumping dredged material is shown in Fig. 109. In this case the diaphragm-type steel sheet-pile cells form a sea wall at the Inland Steel Company’s plant on the Great Lakes. The sand filling was covered with a cap of concrete 9 in. thick.

For the projects of the Tennessee Valley Authority in the United States a number of cofferdams has been used, mostly of very large size and of cellular construction. From Table XIV the speed of the driving will be noted. The sheet-piling was re-used on each of the projects for each of the three successive

<table>
<thead>
<tr>
<th>Site of lock</th>
<th>Total square feet driven</th>
<th>Average length of piling (ft.)</th>
<th>Average depth of overburden (ft.)</th>
<th>Diameter of main cells (ft.)</th>
<th>Average time per cell for construction* (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pickwick</td>
<td>388,988</td>
<td>51'0</td>
<td>14</td>
<td>58'89</td>
<td>21'00</td>
</tr>
<tr>
<td>Guntersville</td>
<td>240,654</td>
<td>39'8</td>
<td>6</td>
<td>42'97</td>
<td>19'00</td>
</tr>
<tr>
<td>Chickamauga</td>
<td>288,880</td>
<td>43'0</td>
<td>13</td>
<td>47'75</td>
<td>29'44†</td>
</tr>
</tbody>
</table>

* For setting template, setting and driving piling, removing template, moving equipment, and delays.
† The rock bottom was extremely uneven and in some cases piling was driven 30 ft. below the normal rock bottom, which made necessary a considerable amount of splicing.
cofferdam stages. The cells were filled with sand and gravel. It is of interest, for their relative values, to quote the average costs for driving and extracting the piling at the Pickwick Landing dam. The make-up of the cost of the three cofferdam stages of that site was

<table>
<thead>
<tr>
<th></th>
<th>Per cent.</th>
<th>Dollars per ton</th>
</tr>
</thead>
<tbody>
<tr>
<td>Handling and hauling</td>
<td></td>
<td>7</td>
</tr>
<tr>
<td>Templates</td>
<td></td>
<td>13</td>
</tr>
<tr>
<td>Setting piling</td>
<td></td>
<td>29</td>
</tr>
<tr>
<td>Driving</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td>Pile pulling</td>
<td></td>
<td>32</td>
</tr>
</tbody>
</table>

Examples of cofferdams for bridge foundations (Figs. 110 and 111) show the reconstruction of Waterloo Bridge and a bridge at Baghdad.

Fig. 109.—Filling Diaphragm Type Cellular Bulkhead with Dredged Sand.

In the main reconstruction of Dunkirk harbour in 1947 to 1954, following extensive damage during the war, four cellular cofferdams (8.7) were used to enable the water level to be reduced for the repair of the lock and dock walls. Steel sheet-piles having an ultimate strength in tension across the interlock of 100 tons per foot of pile were used.

The assumed properties of the soil penetrated by the sheet-piling were as follows.

<table>
<thead>
<tr>
<th></th>
<th>Angle of Internal Friction</th>
<th>Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation soil:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mud saturated with water</td>
<td>10 deg.</td>
<td>1.1</td>
</tr>
<tr>
<td>Sand</td>
<td>25 deg.</td>
<td>1.0</td>
</tr>
<tr>
<td>Dry sand</td>
<td>30 deg.</td>
<td>1.5</td>
</tr>
<tr>
<td>Filling:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry sand</td>
<td>30 deg.</td>
<td>1.5</td>
</tr>
</tbody>
</table>
CONSTRUCTION OF COFFERDAMS

For filling of the cells, grey sand from the Dunkirk dunes was used; this material contained about 10 per cent. of gravel and shells, but otherwise was medium to fine and had an angle of internal friction of 30 deg. when dry. The piles were mostly 56 ft. long and driven by two pile-frames on floating pontoons with automatic hammers; driving was assisted by water jetting with a pressure of 100 to 150 lb. per square inch. Importance is attached in the description of the work to the need of very strong templates or guide walings, and the advantages of pitching and setting all piles and driving them in stages. This enabled all the cells, with one exception, to be closed without the need of special closure piles. For this type of sheet-pile, with a separate clutch, the interposition of a block of hardwood was found beneficial to avoid too much damage to the heads of the piles. The piles were driven in pairs where possible. As the cells could not be filled immediately after the driving was complete, since it was necessary to keep a partial balance of pressure on the division walls, the completed but unfilled cells were liable to damage from waves. Damage from one storm was corrected by water jetting and tension applied at the top of the sheeting. Mention is made that the maximum penetration that can be obtained by water jetting in the Dunkirk sand is 39 ft. Most of the sheet-piles were extracted and used again more than once and some three times in the course of the work. Driving

Fig. 110.—Cofferdam for Pier of Waterloo Bridge, London.
and extraction was said to tend to open the grips of the clutches, with some possible loss of strength, but not sufficient to prevent the piles being used for other work. For work exposed to wave action, a web thickness of $\frac{3}{8}$ in. is suggested as the desirable minimum, but otherwise a thickness of $\frac{1}{8}$ in. is sufficient. Based on costs in early 1953 and the numbers of sheet-piles driven and total penetration as given below, the cost of pitching and driving varied between $115$ francs to $646$ francs per square foot of penetration, but the proportion of the

![Fig. 111.—Cofferdam for Pier of Bridge in Baghdad.](image-url)

cost of the pitching and driving to the total cost, including local transport, reconditioning extracted piles, locating obstacles, and temporary work such as templates, was fairly consistently about half of the total cost. The average rate of extracting the sheet-piles was $1400$ sq. ft. of embedded area per day of eight hours, or approximately $41$ sheet-piles, and the cost, in this case in 1950, was about $150$ francs per square foot (including $50$ per cent. social charges), which was $43.5$ per cent. of the total cost of that operation including transport, preparation of extracting equipment, and mounting the floating crane.
Movable Cofferdams.

Where the water is deep and a number of similar foundations has to be constructed, as in the case of the Storstrom bridge \(^{(8,8)}\) in Denmark, an alternative method (Fig. 112) may be adopted by which the cofferdam itself may be moved from pier to pier and form a template for the driving of the bottom sections of

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![Diagram](image)

**Fig. 112.** Method of Using Movable Cofferdams, Storstrom Bridge, Denmark.

the sheet piling. In this case, after construction of the foundations in the way indicated, the water was allowed to enter and the top section floated away to the next pier to be constructed.

The bridge over the Yangtze river at Hankow, \(^{(8,9)}\) which was completed in 1957, has nine main spans of 420 ft. and carries a six-lane road at the upper level and two railway tracks below. The maximum flow of the river is 2,700,000 cu. ft. per second with a velocity up to 7 m.p.h. The river bottom varies from fine sand, overlaying coarse sand and gravel, to rock and is subjected at times
to considerable scour. The bedrock is limestone, marl or shale with a dip of 70 deg. and steeper. The piers are 55 ft. diameter and each is supported on 30 to 35 concrete cylinders arranged in concentric circles. The cylinders are 61 in. in external diameter, 4 in. thick, and are made in sections generally 30 ft. long. Each section is reinforced with forty-four \( \frac{1}{2} \)-in. bars vertically and horizontally at 6 in. pitch. The end of each section has a steel collar to which the reinforcement is welded, with a flange for forty-four bolts to connect to the next section. The sections were sunk by a vibratory pile-driver and assisted by four 3-in. jetting pipes attached to the outside and one on the inside. The supply to each jet was 360 Imperial gallons per minute at a pressure of 175 lb. per square inch. The vibratory pile-driving machine followed Russian practice by having two axles
rotating at high speed in opposite directions, each with eccentric weights. The number of the axles and the weights may vary and also the speed of rotation of the axles, say from 408 up to 1000 r.p.m., so that vibrating forces may be 19 to 132 tons.

A typical vibratory pile-driver comprising two vibrators developing a downward force of 320 metric tons is shown in Fig. 113. It has been stated that this method is of particular advantage in waterlogged sands and gravel and in driving steel sheet-pile, but is of less advantage in cohesive soils. Large vibrators may have up to eight eccentric weighted rotating axles, some of which rotate at half the speed of the others, thereby giving a downward component larger than the upward one. The diagram in Fig. 114 shows how the exiting force of a six-axle vibrator varies with the positions of the weights. A vibrator weighing 11-75 metric tons creates a maximum downward force of 90 tons when powered by a 155-kw motor. Larger vibrators have been built to enable the sinking of hollow thin reinforced concrete cylinders of 16-5 ft. diameter to a depth of 115 ft.

At the Hankow Bridge, to deal with the steep dip of the rock surface, concrete was placed in the bottom of the cylinder when one edge first struck the rock so as to seal the cylinder. The socketing of the cylinders 7 ft. to 20 ft. into the rock according to circumstances was then done by boring through the concrete and on into the rock. With boring bit of cruciform pattern, 51 in. wide and weighing 4-4 tons, falling 2 ft. to 3 ft., thirty to forty times per minute, the time to penetrate 10 ft. is reported to have varied from 20 hours to 28 hours.

A cage of reinforcement, of 43 in. diameter with eighteen to twenty-four 14-in. bars vertically and 1-in. binding, was lowered into the sockets to project well up into the cylinders. The cylinders were filled by tremie with concrete up to the bases of the piers.

River Crossings.

For any type of permanent construction across a river and which is continuous below water-line, for example dams and pipe crossings, several cofferdams are used consecutively, so that the permanent work is carried out in stages without undue restriction to the river flow. Often a large part of the materials for the cofferdam of the first stage may thus be re-used in the third stage. As the stages must overlap, if there are only two stages, no re-use is possible without an interruption in continuity of the permanent construction, starting from the commencement of extracting the sheet piling until the dewatering of the second stage cofferdam.

For small contracts it is usually better to keep the work continuous than to re-use the piling and bracing, while on large contracts both continuity and re-use of the cofferdam materials are possible by dividing the work into three or more stages. As an example, Fig. 115 shows the overlap of successive steps in the case of the Ice Harbour Dam on the Lower Snake River, Washington.

The procedure is only slightly modified in the case of dams if part of the cofferdam sheeting is left in to form a permanent cut-off against underseepage, but in the case of rivers which flood seasonally the permanent work may in any case need to be carried out in separate operations and the cofferdam stages arranged to suit. Protection against scour is important as the following example shows.
Cofferdam on the River Nile.

Irrigation requirements on the river Nile have entailed the use of numerous large cofferdams, or sudds as they are somewhat ambiguously known, on the river Nile. Most sudds have consisted of sand banked up to enclose the area concerned, and it is only in recent years that sheet-piling has been used in addition for the temporary construction, as distinct from sheet-piling driven across the river both upstream and downstream of barrages to form a permanent cut-off against underflow.

Fig. 116 shows the sudds for the several stages of the work in the construction of the Nag Hammadi barrage, the whole project involving several seasons' work, the sudd being removed at the end of each season and the next sudd commenced when the ensuing low-water period occurred. On this site the sub-soil was entirely sand.

Quoting from an account of the work by Mr. A. R. Ellison\(^8,11\) regarding the cofferdam construction:

"It was intended to construct each season's sudd [Figs. 116 and 118] of steel sheet piling supported on each side by sand-bags tipped to a depth of 2·5 metres above bed-level and further supported on the inside by tipped excavation and on the outside by pumped excavation. This design was adhered to in constructing the first season's sudd, but for the other sudds the use of sand-bags, except in special circumstances, was discontinued, the piles being supported on the inside by tipped rubble, and sand being pumped against the outside only where it was thought desirable to increase the length of water-travel. Larssen interlocking piles, ranging in length from 11 to 15 metres, were used. The designed minimum penetration was 4·5 metres and the
Fig. 116.—The Nag Hammadi Barrage on the River Nile. [The north (downstream) side is at the top of the plan.]
CONSTRUCTION OF CUPPERDAMS

By depositing some of the tunnel's excavation and by the removal of the first season's spoil, the Walshe Pool was raised to the southern low of the dam area and the southern end of the Walshe Pool was removed. The only way to raise the southern low of the dam area was to deposit spoil on a level surface. The spoil was then piled on a level surface and the level surface was raised by depositing spoil on the northern low of the dam area and by depositing some of the tunnel's excavation. The spoil was then piled on the northern low of the dam area and the northern low of the dam area was raised by depositing spoil on a level surface. The spoil was then piled on a level surface and the level surface was raised by depositing spoil on the southern low of the dam area.
The work involved, inter alia, some 181,000 cu. m. of excavation, 2033 reinforced concrete piles, and 1442 tons of steel piles.

The total width of the Nile is about 2700 ft. Fig. 117 (pages 160 and 161 shows a cross-section through the barrage and Fig. 119 gives cross-sections through the upstream and downstream sudds. A general view of the work is seen in Fig. 120. The two lines of sheet-piles on the upstream sudd were provided as a precautionary measure owing to the possible consequences of any failure of the existing structure. Quoting from a description of this work by Mr. J. E. Bostock (8,13) with regard to the hydraulic gradients in the sudds and how they were affected by the sheet-piling:

"Experiments were made to determine the hydraulic gradients in the sudds and how they were affected by the sheet-piling. The following results were obtained:

(a) Sand impregnated with silt gave gradients as steep as 1 in 1.3 on the upstream side outside the piling. The sand on the downstream side, which was coarser and cleaner, produced gradients of 1 in 7 or steeper.

(b) The gradients of the creep down and up the sheet piling ranged from 1 in 7 to 1 in 15, depending on the depth of the piles in the river bed as compared with the depth of the sand filling.

(c) The inner row of piling on the upstream side definitely ponded up the water and, by reducing the velocity of the flow, decreased the possibility of movement in the solid materials and produced a safer sudd.

On completion of the season's work the sudd was slowly re-watered and pile extraction commenced. The greater part of the sand filling of the sudds was washed away by the following high-river flow."

The steel sheet-piles for the sudds were Larssen No. 2 piles driven by McKiernan-Terry hammers of sizes Nos. 6 and 7, and exceptionally No. 9, the piling was driven to a depth of roughly 10 metres below the river level and about 5 metres into the river bed. On the downstream side a single row of steel piling was driven to a similar depth below the river level. At the forward end of the
UPSTREAM SUDD, LOOKING WEST.

DOWNSTREAM SUDD, LOOKING WEST.

Fig. 119.—Typical Section of Sudds, Assiut Barrage.
sudds, for the first three seasons the outer row of Larssen piling was driven hard up to the edge of the old floor of the barrage. The last few piles were driven in the materials which were impregnated by the cemented cut-offs described previously. A short length of curtain piling was incorporated, running longitudinally with the old floor. The back ends of the second, third, and fourth seasons' sudds were rendered tight by the planting of the outer row of piles in chases; these chases were purposely left on top of the new floors and were filled with a sand and bituminous mixture.

Roughly one-third of the driving was through rubble consolidated with silt, and the remainder into the sand of the river bed. The extraction of the 3720 piles occupied roughly 60 days and nights. The total length of extraction was about 35,500 metres, which allows for the sand filling of the sudds deposited after driving. The filling outside and inside the piling was carried out by suction dredgers and other means. In some cases the salient corners were protected against scour by tipped rubble. The dewatering was carried out slowly in order to allow the sudds to drain out gradually, after which there was little seepage water throughout the sudds.

**Closure Operations in Rivers.**

Figs. 121 and 122 show the method of maintaining sheet-piling plumb during closure operations for a dam on the Mississippi river at Alton, Illinois. Complete details of the work are given in a book on cofferdams, to which the reader is recommended for other examples of large cofferdams on the Mississippi river. Some method of this type is essential when effecting the closure of the sheeting with the velocity of the flow increasing as the channel becomes narrowed. Attention is drawn to the usefulness of dredgers of large capacity in depositing sand on the river bottom along the upstream side of the sheeting when making closures in these circumstances.
PLAN OF CLOSURE TRESTLE AND FALSEWORK GUIDE.

METHOD OF SETTING STEEL SHEET-PILING IN SWIFT WATER.

Fig. 121.—Method of Effecting Closure of Final Cofferdam Stage.
In effecting the closure indicated in Fig. 121, use was made of three dolphins each of seven 70-ft. piles placed 100 ft. on the upstream side of the closure, with

![Image](image-url)

**Fig. 122.—Closure Operations for Final Cofferdam Stage, Lock and Dam No. 26, Upper Mississippi River.**

(See upper part of illustration and also Fig. 121.)

a 3-in. wire cable and turnbuckles connecting to each bent of the closure trestle. The closure trestle was placed upstream of the closure, because of the reduced tendency for scour compared with the alternative downstream position. In this case suction dredgers were used, obtaining material from several points several hundred feet upstream and depositing it along the line of the upstream arm of the cofferdam.

**Closure Cofferdam on the St. Lawrence.**

Closing the south channel of the St. Lawrence river where it races around Long Sault Island called for the use of an unusual rock-filled timber crib and sledge to cut off the flow and permit construction of an ordinary steel sheet-pile cellular cofferdam. The dam extends 3290 ft. from the U.S.A. mainland to the upstream end of Barnhart Island, above which the St. Lawrence is split into
two channels by Long Sault Island. The plan was to block the South Channel with cofferdam A and divert its flow through a cut across the island into the main stream. Cofferdam B which extended from the tip of Long Sault Island to the mainland would then enclose the lower end of the 25-acre area where the first part of the Long Sault control dam was built. While the diversion cut was being excavated, four cells of cofferdam A were constructed, two adjoining each shoreline. The interlocking steel sheet-pile cells, each 59 ft. 8½ in. in diameter, were driven by usual methods with crawler cranes working on earth berms from each shore. These installations narrowed the width of the channel between the cells to 265 ft., and resulted in a velocity of about 10 ft. per second. The next operation was to close the opening between the cells so that the four remaining cells of cofferdam A could be placed, as it was impossible to build the cells in the raging stream. Studies indicated that the closure should be designed for a maximum differential head of 16 ft., a flow of 35,000 cu. ft. per second and final velocities up to 24 ft. per second.

The threat of ice jams prevented building a trestle and dumping rock in a weir-type closure and also because of the erratic rocky and boulder-strewn bottom of the channel. Also no large rock was available locally. A steel closure crib and sledge was fabricated on the mainland shore. The sledge was 115 ft. long by 47 ft. wide and had three runners of 24-in. 100-lb. steel beams. The runners were hinged in the middle and bent up at one end so they would travel over the uneven river bed. Six 24-in. 100-lb. beams 29 ft. long, which were erected upright and braced to the runners, formed guides into which heavy screens could be dropped to hold a rock ballast in the swift stream. Wooden cribs 19 ft. wide, 45 ft. long and 24 ft. high, made of poles, were built on each end of the sledge, and were designed to hold about 1000 tons of rock to ballast the structure at midstream in the fast-running water. When the cribs were installed on the sledge runners, there was an opening 60 ft. wide between them, and was the final closure gap. To move the sledge, a cable was fastened to each of the three runners; the middle cable was 1½ in. in diameter and had a breaking strength of 110 tons; the other two were 1½ in. in diameter and had a strength of 90 tons. Each cable was carried across to the island where diesel cranes were used for power through sets of blocks attached to 100-ton deadmen. Lateral drift of the sledge was maintained by a 1¼-in. cable leading upstream from the forward corner through a set of blocks to a bulldozer. A drag-cable was attached at the rear to prevent the structure from sliding too fast down the icy riverbank.

Pulling the sledge a distance of 150 ft. over the rocky river bottom to its intended position in the deepest part was an easy operation. When the sledge was positioned, the landward crib was filled with rock and a crawler-crane used it as a base to reach across and fill the other crib. The cables were removed and work began on a dyke between the eighth cell and the crib on the island side of the sledge. Rock had been previously ferried across the channel and stock-piled. The rock core for the island dyke was initially dumped up to an intermediate level about halfway to the crib, thereby allowing about 30,000 cu. ft. of water per second to pass before the diversion cut across the island was unplugged. For the first half of the dyke from the island, rock in pieces up to 1 ton in size was used. As dumping progressed into the other half, the maximum size was increased to 2 tons. The larger rock was dumped on the upstream
corner of the fill to deflect solid water and allow the use of smaller material to extend the berm.

One week after the last plug had been removed from the diversion cut, the dyke extended to the crib. To bring the top of the dyke up to the final level and give it ample freeboard for maximum flow, sand and gravel were placed to a depth of 9 ft. above and upstream of the rock core of the dyke; this also reduced the permeability of the dyke. Final closure was achieved by placing fabricated screens between the uprights on the sledge. The screens were of 12-in. side channels and 12-in. beam cross-members at 5-ft. centres, with vertical 5-in. rods at 12-in. centres. The screen for the central opening, immediately above the hinge in the sledge runner, was designed so that its width was variable. Half pieces of sheet-pile were used as side members and interlocked with the other half pieces of pile welded to the vertical beams. The cross-members, which were lengths of 1-in. steel cable at 12-in. centres, were spaced by wrapping and stapling them to timber poles. Placing the rock filling upstream of the screens was simple and speedy, the greatest size needed being only 12 in. by 15 in. After the closure, the remaining four cells of the cofferdam were installed in the still pool formed by the dyke.

Avoiding the Need for Cofferdams.

Where it is not necessary to work below water in the dry, there are several ways of forming load-bearing substructures as described in the following pages.

Some years ago the construction of piers had generally to be done in the dry, but successive improvements in plant and constructional practice have made other methods more competitive as to cost. One method which is applicable where the bed of the waterway is too soft for a direct bearing so that piling is needed, is to place under water precast concrete cylinders on the piles and then fill the interior with concrete placed by tremie. The interior can also be filled with coarse aggregate and then made into concrete by the injection of colloidal cement and sand through one or more pipes previously set in position. The precast cylinder can be in short sections placed one at a time by a crane, in which case three piles may need to be driven accurately previously to form a template. If the cylinder is made on land of the full length required, it may have one or both ends sealed and be floated into position. Prestressing the cylinder may give it the necessary strength to sustain handling or launching. If the bed of the waterway is rock, the use of cylinders in this way is still an alternative to a cofferdam. Although with cofferdams or cylinders there is the problem of preparing a levelled bearing surface, with a cofferdam there is the difficulty of making watertight the irregular meeting surface of the dam and the rock; with cylinders this is replaced by the problem of preparing the levelled bearing surface under water.

Safety of Working in Cofferdams.

At the time of going to press draft regulations relating to the safety of cofferdams and caissons have been issued. The matters dealt with include means of escape, supervision, and inspection before men work in cofferdams.

Note.—Bibliographical references for Chapter VIII are given on page 259.
CHAPTER IX

THE PRINCIPLES OF CYLINDERS AND CAISSONS

Where it is not feasible to use the cofferdam method, say, because of boulders in the soil, or where it would not be economical to do so because the foundation required is small in plan area in relation to the depth of water, one or other variation of the caisson method is likely to be suitable. In particular, cylinders and caissons are used where a cofferdam could not successfully be dewatered, say because of the depth of water and the type of soil that has to be penetrated, or because of the permeability of the soil below the foundation level. By all variations of the caisson method it is the shell of the permanent foundation that is sunk to reach the bearing soil, and the various methods form alternatives to piling or to building a spread footing within the protection of a cofferdam.

Piling will generally be the most economical if the loads are small, and where the total load is large if the load is distributed over a sufficiently large area, and if in either case the depth to the bearing soil does not exceed 80 ft., or in special cases 110 ft. if prestressed concrete piles are used. Cofferdams are economical for soils in which sheet-piles can be driven satisfactorily and under-seepage limited if the load is more concentrated and the depth below standing water is less than about 40 ft. There are also cases where piles are not suitable.

The useful range for cylinders and caissons is generally covered by the following cases:

(a) Concentrated loads of bridge spans.
(b) Foundations for heavy loads; for deep-water quays and other heavy engineering structures having large or concentrated loads, particularly where the sub-structure also resists substantial horizontal forces.
(c) Any heavy foundations where obstructions or boulders would prevent the successful driving of bearer piles or of sheet-piles for a cofferdam.
(d) Heavy foundations that are deeper than about 40 ft. below standing water level.
(e) Any foundations that pass through soil that will flow into open excavations and where the cofferdam method is not feasible.

These cover the majority of cases where either cylinders or caissons are likely to be more economical than piling or the cofferdam method, or a combination of the two.

Both cylinders and caissons are frequently surmounted by a cofferdam or temporary caisson within which the permanent pier is built upon the cylinder or caisson and the cylinder or caisson shortened so as not to project above low water level, the cost of the whole pier being thereby reduced.

Apart from this variation and mixtures of methods applicable in special cases, by treating cylinders as a special type of caisson the following become the principal types:

(a) Box caissons in which the bottom is closed.
(b) Open caissons in which the top and bottom are open.
(c) Pneumatic caissons in which the top is closed and the working chamber is under air pressure.

Other methods such as open wells are also shown in Fig. 123, so that the alternatives to piling are grouped together. Whether the construction is circular in plan or some other shape is more dependent upon the load to be carried and the external forces than a distinction between the methods. The difference between cylinders and open caissons is that cylinders have single walls and are generally sunk by the addition of kentledge, by impact, or by jacking down, while caissons sink either under their own weight or by the addition of concrete or other permanent filling. The distinction between cylinders and caissons is not definite in technical literature or in practice and frequently, especially abroad, cylinders, as just defined,
are described as caissons. It is felt, however, that the preceding definition could usefully be more widely adopted. When cylinders are sufficiently small in diameter to be driven by impact and of considerable length these become more truly tubular piles. Here they will be treated as cylinders if the ends are open and as piles if they are driven with a closed shoe.

As shown in Fig. 124, caissons are more easily kept vertical during sinking than cylinders, whatever way the latter are sunk, and for this reason staging is invariably used to keep cylinders plumb while sinking. In addition, while cylinders have the one access shaft, with open caissons there may be a number, use generally being made of this in excavating to keep the caisson vertical during sinking. When caissons are sunk through water, guide piles are often used to ensure their exact position, but it is still necessary to provide measures so that the caisson sinks vertically. The guide piles will assist in siting the caisson before sinking commences, but extremely little afterwards since they are easily pushed aside; when the caisson can be easily sited without them they are omitted. A timber piled staging is usually provided to site the caisson and from which it may be lowered on to the waterway, and this staging is usually made large enough to support cranes and concreting plant according to the type of caisson and the method of sinking being used.

LIMITATIONS OF CAISSON METHODS.—There is practically no limit to the depth of open caissons sunk in water, and this method is used for the deepest foundations. With pneumatic caissons, however, the maximum depth is limited by the limit to the air pressure in the working chamber, and is usually taken as 110 ft. below water level.

There is a limit to the minimum diameter of cylinders, or the size in plan of caissons, at which piles become more economical, and because of this cylinders and caissons are seldom economical unless the loads to be supported are either large and concentrated or the depth of the foundations is great.

OPEN-WELL FOUNDATIONS.—Provided the soil to be penetrated can be successfully excavated in the dry, or under conditions where the excavation can
Fig. 125.—Leesh Bridge.
be lined, or will hold up without lining, open wells can be used to obtain foundations on a sub-soil of good bearing value. These conditions are seldom encountered, as often when the soil is good enough to be excavated by the open-well method it is also good enough for the foundation; if it is not good enough as a foundation it is seldom possible to excavate it by this method, and a choice must be made between piles and cylinders. The exception is a clay sub-soil suitable for open well excavation but not for the support of the heavy concentrated loading of high buildings, and the use of this method in the Middle West of the United States originated with these conditions.

Fig. 127.

Indian practice has been based on the use of brickwork for the walls of open caissons from early times and before Portland cement concrete was first used. Reinforced concrete is now tending to be used in place of the brickwork, but if brickwork is used a kerb of reinforced concrete is generally provided. The writer is not aware that the principles for the design have been published, but the reader should refer to a paper by Mr. Salberg and the Indian Railways Standard Code of Practice.⁹¹ It is stated in the latter that support by skin friction should generally be ignored as it is dependent upon there being no scour, which Indian railway engineers have good reason to take into account, and also because it does not usually result in much reduction in the size of the well required. There is no doubt, however, that even poor soil in contact with the embedded part of a well is of definite assistance towards stability, and is not insignificant in resisting seismic forces.

A recent typical example of wells of moderate depth is the piers of the Leesh bridge near Siliguri in North Bengal. Figs. 125, 126, and 127 show respectively the elevation of the bridge, plan and elevation of one of the wells, and a view of

* References thus (⁹¹) refer to Bibliography on page 259.
the piers under construction. In areas subject to seismic forces stability under lateral forces is usually improved by partly filling the wells only.

**Box Caissons.**—If a level bearing surface exists or can be prepared, say, by dredging or divers, or, if on land, say, by excavation (perhaps in conjunction with the well-point system of pumping), then box caissons [Fig. 123 (h)] may be used. The limitations of this method in water are the practical difficulties of preparing the levelled bearing surface, and the need to ensure that stability is not affected by scour. On land a disadvantage is the inability to inspect the bearing surface of the closed bottom. Where rock forms a practically level surface which can be prepared by a diver, or levelled with concrete placed by tremie, the method has been used successfully on numerous occasions. In recent years the principal use of box caissons has been for quay walls, a series of units being sunk to form the line of the quay upon a prepared bed of sand, which may be placed in a shallow trough dredged from the channel bed if the latter is silt or other poor soil.

**Cylinders.**—Open steel or concrete cylinders may be lowered to the bed of a waterway and the enclosed soil excavated by grab until the cylinder reaches a satisfactory foundation [Fig. 123 (f)], or alternatively [Fig. 123 (g)] piles may be driven within the enclosed area without the necessity of the cylinder reaching a hard bearing. In both these methods the interior is usually filled with concrete for the full height. Where for any reason the enclosed soil cannot satisfactorily be excavated under water, it is necessary to use the pneumatic caisson method described later and in Chapter XI.

**Open Caissons.**—The method of sinking an open caisson and grabbing out the soil under water is generally the same as the cylinder method, except that with caissons the walls themselves carry part or all of the load, sometimes by filling compartments with concrete or masonry as sinking proceeds, and open caissons are normally sunk by means of their own weight.

**Pneumatic Caissons.**—Where the soil cannot be excavated through open shafts in the caisson, say because the soil to be excavated is below water level and includes boulders, buried timber, or masonry, or because of unusual properties of the soil itself, it is then generally necessary to use pneumatic caissons, by which the working chamber is under compressed air and the men gain access and the material is removed through vertical access shafts provided with air locks.

Examples of each of the foregoing types are described in Chapters X and XI, and demonstrate how circumstances on the site favour particular methods.

**Depth of Caissons.**

The depth of the caisson should be determined by the minimum height necessary to provide, or support satisfactorily, the weight required to overcome the skin friction expected. When a small penetration only is necessary the caisson will often be kept shallow, and in the case of pneumatic caissons may consist only of the height from the cutting edge to the top of the roof of the working chamber, say some 10 ft. When the penetration is expected to be great, open caissons are usually built up to some 30 ft., or to the full height, before sinking commences. With pneumatic caissons there is the alternative of surmounting
a shallow caisson by a cofferdam, often described as a temporary caisson, within
which the pier may be built up as sinking proceeds and provide the additional
weight necessary, as the total skin friction increases with penetration.

The choice between open and pneumatic caissons is decided by the sub-soil
as previously described, but the height of the caisson will usually be decided by
the method of constructing and placing. If built suspended from temporary
staging, lowering into the water will be commenced with the caisson as small
and light as possible, and the caisson will be built up after it is obtaining support
by buoyancy. If the caisson can be floated into position it will usually be most
economical to construct it to a substantial part of, or to, the full height before
commencing sinking.

**Sinking of Open Caissons.**

Open caissons are usually sunk by excavating by grab through the open cells
while the water level inside the caisson is about the same as that outside. The
caisson is kept vertical during sinking by varying the amount of excavation
between the cells. Alternatively, and more particularly with smaller cylinders
and caissons, the excavation may be made by sand pump, the water that is lost
in this way from the cylinder or caisson being replaced by pumping in clear water.
Most fine soils can be easily extracted in this way, the pipe size being about 5 in.
This method is usually adopted in the case of cylinders that are of too small a
diameter for mechanical excavation.

**Placing the Concrete Seal.**

When the penetration of open caissons is into an impermeable stratum such as
clay, it is sometimes possible to place the concrete in the dry after pumping out
the water in the same way as if the caisson were a cofferdam. There are, however,
the same disadvantages that apply in the case of cofferdams, that is, there will
then be active pressure in the soil tending to lift the floor and in addition, if there
is any doubt about the permeability of the stratum, there is the risk of blows
which may make the method dangerous for placing concrete by hand. The two
usual methods are as follows.

If the caisson can be dewatered successfully the concrete may be dropped
from the surface from concrete skips; on the other hand, if it is not practical
to dewater the interior the concrete may be placed under water by tremie, both
methods being described in examples which follow.

(Placing the concrete seal for pneumatic caissons is referred to on page 228.)

**Cutting Edges.**

The type of cutting edge detail depends upon the method of construction of
the cylinder or caisson, the method of sinking, the soil penetrated, and the soil
into which the cutting edge finally rests while the concrete seal is being placed.
Inward forces may arise due to sinking out of plumb and outward forces due to
the wedging action of the seal upon a slope to the inside. If the sinking is entirely
through soft soil a detail as Fig. 128 (e) may be suitable in which the seal supports
a cylinder or caisson by a horizontal surface of contact, and this type of con-
struction has been used for pneumatic caissons. The detail (a) is typical of earlier
Fig. 128—Examples of Cutting-edge Details.
practice for open cylinders of steel construction, for example, as used for the caissons of the original Forth Bridge. The detail (d) of a pneumatic cylinder used on the Columbia river bridge at Trail, British Columbia, is not so typical of present practice as (b), which shows the detail used for the open caissons of the Delaware river suspension bridge,\(^{9,12}\) or (f), which has a cutting edge suitable for building up from a land surface. Where a brick lining is used for a steel open well or cylinder, a suitable cutting edge may be that shown at (e), in which case the inside strokes of plates are omitted, but the detail shown at (c) is more suitable for average conditions of sinking. The cutting edge detail used for the first Narrows suspension bridge at Vancouver, British Columbia,\(^{9,30}\) completed in 1938, was somewhat similar to the detail (f), having an 8-in. width of bearing of which 6 in. was made up of a 6-in. by 6-in. by \(\frac{3}{4}\)-in. kerb angle.

**Design of Caissons.**

In the past large caissons have been constructed without cross walls, and are often circular in plan. Generally, however, present practice is to construct open caissons with cross walls and longitudinal diaphragm walls dividing the whole caisson into cells usually not smaller than 15 ft. square and not larger than 20 ft. square. This applies also to very large and deep caissons.

The more obvious requirements in the design of caissons do not involve departures from constructional design for other types of sub-structure. Thus the caisson walls are designed to withstand the lateral pressure of water and submerged soil at the final depth to which they are expected to be sunk, and the walls are designed for the final permanent direct compressive load of the complete construction, the walls then being treated as columns restrained in direction by the cross walls. The compressive stresses in the walls are then transferred to the soil through the concrete seal, the load that can be taken by the cutting edges being negligible in comparison with that which must be transmitted through the seal.

In the case of concrete open caissons divided into cells, the moments caused in the walls by the active pressure of the soil and water may be obtained by the method of moment distribution, treating all corners as monolithic and reinforcing them for these moments. The moments taken by the cross walls will be nil theoretically, for square cells for this type of loading, but in the case of unequal, soil pressure distribution, moments in the cross walls may be large and should be provided for. An example of the moments in the walls of a typical reinforced concrete caisson subjected to the active pressure of fine sand soil and water at a depth of 100 ft. is shown in Fig. 129.

In the case of pneumatic caissons or any other type of caisson having a horizontal diaphragm a few feet only above the level of the cutting edge, this diaphragm or the roof of the working chamber may be subject to a large part of the reaction of the bearing soil, usually necessitating stiffening by deep beams capable of safely taking the resulting shearing forces and bending moments.

The preceding remarks apply also to steel caissons having double walls in which permanent concrete filling is placed during sinking. With caissons in which the walls taper to the cutting edge and the concrete seal bears against the sloping surface, the tapering portion of the wall will be subject to the possibility of moments due to the reaction transmitted through the concrete seal tending to
burst out the tapering portion of the wall. With circular reinforced concrete caissons this may be dealt with by providing reinforcement in ring tension.

In some types of concrete caisson, level bearing surfaces are provided for supporting the walls from the concrete seal by forming the slope in a succession of inverted steps, while with steel caissons the cutting edge is often stiffened with brackets and a flat contact surface provided for the seal immediately under the concrete fill between the inner and outer strakes.

In addition to these more general requirements, it must be expected that the caisson will not always be sinking truly vertically, and a considerable portion of the total support during sinking may be provided by the skin friction of the soil acting on parts only of the exterior of the caisson. These forces can be very large and may easily be sufficient to break off the cutting edge of a concrete caisson.

Occasionally the timber formwork of concrete caissons has been left in place during sinking, and this is in fact practically necessary if the caisson is being built up in stages and has to be sunk with the concrete not completely matured, since the friction forces developed would otherwise be sufficient to separate the lifts by a failure of the concrete in tension and insufficient bond having developed with the vertical reinforcement.

In the case of pneumatic caissons the roof of the working chamber must be designed to withstand the uplift force due to the maximum expected air pressure in the working chamber, and also for the downward load due to concrete filling to the cells if this is used either for assisting sinking of the caisson or as part of the necessary permanent construction, on the assumption that it is not conceivable that the roof would not be supported immediately underneath by water pressure or by a tightly-placed concrete seal.

Where the air locks are mounted on top of the caisson and reinforced concrete forms the lining to the access shaft, the concrete construction is designed to resist the circumferential tension due to the air pressure in the shaft, and also provide for resisting the uplift pressure tending to blow the air lock and its anchoring bolts from off the top of the shaft.

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**Fig. 129.**—Example of Bending Moments in Caisson at a Depth of 100 ft. below Water Surface.
Caisson Friction.

It would be expected that the skin friction resisting the sinking of caissons would correspond very closely to the calculated resistance obtained by use of the formula

\[ F = \frac{\mu wh^2}{2} \left( \frac{I - \sin \phi}{I + \sin \phi} \right), \]

that is to say, using the Rankine formula for the active pressure for the depth, in conjunction with the coefficient of friction \((\mu)\) for the particular soils penetrated.

![Caisson Sinking Effort in Sand](image)

Fig. 130.—Relation of Sinking Effort in Sand to Rankine Values of the Active Pressure with Coefficient of Friction \((\mu) = 0.3\).

For granular soils only it can be seen from Fig. 130 that an apparent value of \(\mu\) can be found which suits actual recordings of sinking effort in sand. For cohesive soils, however, numerous cases of determining the total skin friction show these forces to be generally less related to the penetration than might be expected. It may be that the skin friction is relatively greater at shallow depths because the cohesion of the soil near the surface is affected by the site operations and the active pressure in the soil corresponds to the natural pressure of undisturbed soil, which is greater than the active pressure in the limiting case of equilibrium considered by Rankine, while it is to be expected that the coefficient of friction is higher than the low values conservatively used when considering the stability of structures against sliding. At greater depths, however, it is possible that the cohesion in the soil is not overcome by plastic flow for some time after the caisson is sunk, and little active pressure is developed until some time later. Unfortunately, although technical literature contains a great number of total values for the skin friction, in most cases the penetration is through several types of soil; in other cases insufficient information is available on the depth of penetration at the time of the test. Fig. 131 prepared by Sir Robert Gales gives the sinking effort for a selection of caissons for bridge foundations sunk in sand.
During the construction of a bridge over the Mississippi at New Orleans, where the main caissons were sunk 170 ft. below water level through silt, sand and clay, the weights available to induce sinking in the two caissons ranged from 730 lb. to 860 lb. per square foot of surface, and these weights could be increased, by pumping down, to 1340 lb. and 1750 lb. per square foot respectively. Even then some trouble was experienced in reaching the required depths, and serious blows occurred before the caissons reached their final levels.

Fig. 131.—Sinking Effort Diagram for Open Caissons sunk in Sand.

The exploratory caissons for the cut-off wall of the Merriman dam, briefly described later, were sunk up to 160 ft. through alternately impervious and pervious strata, the latter including sand, glacial till, and boulders. The minimum skin friction developed was 570 lb. per square foot and the maximum 924 lb. per square foot calculated at times when the caisson was moving. These caissons had a tendency to hang up even when the excavation had freed the cutting edge, and when working under air pressure it was often necessary to blow the caisson suddenly in order to start sinking.

With alluvial soils, in which there is normally a fair proportion of particles of colloidal size, the skin friction developed in sinking cylinders and caissons may show a wide variation on the same site, probably due partly to variations in the
## Table XV.—Skin Friction in Sinking Cylinders.

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of caisson</th>
<th>Method of sinking</th>
<th>Materials penetrated</th>
<th>Average skin friction (lb. per sq. ft)</th>
<th>Depth below low water in feet</th>
<th>Area of base in square feet</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>MAINLY GRANULAR SOIL.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Cast iron</td>
<td>Open excavation</td>
<td>Sand</td>
<td>250</td>
<td>60</td>
<td>125</td>
</tr>
<tr>
<td>2</td>
<td>Do.</td>
<td>Do.</td>
<td>Do.</td>
<td>325</td>
<td>60</td>
<td>125</td>
</tr>
<tr>
<td>3</td>
<td>Timber construction</td>
<td>Do.</td>
<td>Do.</td>
<td>450</td>
<td>30</td>
<td>1,300</td>
</tr>
<tr>
<td>4</td>
<td>Steel construction</td>
<td>Pneumatic</td>
<td>Sand, boulders</td>
<td>450</td>
<td>68</td>
<td>2,700</td>
</tr>
<tr>
<td>5</td>
<td>Timber construction</td>
<td>Do.</td>
<td>Sand</td>
<td>540</td>
<td>75</td>
<td>1,700</td>
</tr>
<tr>
<td>6</td>
<td>Do.</td>
<td>Do.</td>
<td>Do.</td>
<td>650</td>
<td>90</td>
<td>1,200</td>
</tr>
<tr>
<td>7</td>
<td>Do.</td>
<td>Do.</td>
<td>Sand, boulders</td>
<td>660</td>
<td>101</td>
<td>2,100</td>
</tr>
<tr>
<td><strong>MIXED GRANULAR AND COHESIVE SOILS OR SILT.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Steel</td>
<td>Open excavation</td>
<td>Mud, sand</td>
<td>450</td>
<td>65</td>
<td>1,300</td>
</tr>
<tr>
<td>9</td>
<td>Masonry</td>
<td>Pneumatic</td>
<td>Sand, mud</td>
<td>205</td>
<td>40</td>
<td>75</td>
</tr>
<tr>
<td>10</td>
<td>Timber construction</td>
<td>Do.</td>
<td>Silt, sand, mud</td>
<td>310</td>
<td>75</td>
<td>2,550</td>
</tr>
<tr>
<td>11</td>
<td>Cast iron</td>
<td>Open excavation</td>
<td>Gravel, clay</td>
<td>240</td>
<td>60</td>
<td>125</td>
</tr>
<tr>
<td>12</td>
<td>Do.</td>
<td>Do.</td>
<td>Sand clay</td>
<td>250</td>
<td>75</td>
<td>225</td>
</tr>
<tr>
<td>13</td>
<td>Wrought iron</td>
<td>Do.</td>
<td>Do.</td>
<td>285</td>
<td>140</td>
<td>1,000</td>
</tr>
<tr>
<td>14</td>
<td>Cast iron</td>
<td>Do.</td>
<td>Sand, clay, gravel</td>
<td>300</td>
<td>100</td>
<td>125</td>
</tr>
<tr>
<td>15</td>
<td>Steel construction</td>
<td>Do.</td>
<td>Silt, sand, clay</td>
<td>375</td>
<td>55</td>
<td>190</td>
</tr>
<tr>
<td>16</td>
<td>Cast iron</td>
<td>Do.</td>
<td>Silt, mud, clay</td>
<td>390</td>
<td>75</td>
<td>100</td>
</tr>
<tr>
<td>17</td>
<td>Steel construction</td>
<td>Do.</td>
<td>Silt, clay</td>
<td>450</td>
<td>60</td>
<td>700</td>
</tr>
<tr>
<td>18</td>
<td>Do.</td>
<td>Do.</td>
<td>Silt, clay, sand</td>
<td>450</td>
<td>60</td>
<td>1,200</td>
</tr>
<tr>
<td>19</td>
<td>Iron construction</td>
<td>Pneumatic</td>
<td>Sand, gravel, clay</td>
<td>450</td>
<td>65</td>
<td>200</td>
</tr>
<tr>
<td>20</td>
<td>Steel construction</td>
<td>Do.</td>
<td>Clay, sand</td>
<td>275</td>
<td>60</td>
<td>150</td>
</tr>
<tr>
<td>21</td>
<td>Do.</td>
<td>Do.</td>
<td>Sand, clay, gravel</td>
<td>350</td>
<td>100</td>
<td>1,200</td>
</tr>
<tr>
<td>22</td>
<td>Do.</td>
<td>Do.</td>
<td>Sand, clay, boulders</td>
<td>400</td>
<td>48</td>
<td>1,925</td>
</tr>
<tr>
<td>23</td>
<td>Timber construction</td>
<td>Do.</td>
<td>Clay, sand, gravel</td>
<td>400</td>
<td>95</td>
<td>4,300</td>
</tr>
<tr>
<td>24</td>
<td>Do.</td>
<td>Do.</td>
<td>Sand, gravel, clay</td>
<td>425</td>
<td>55</td>
<td>1,500</td>
</tr>
<tr>
<td>25</td>
<td>Timber</td>
<td>Do.</td>
<td>Silt, clay, gravel</td>
<td>500</td>
<td>75</td>
<td>1,800</td>
</tr>
<tr>
<td>26</td>
<td>Timmer construction</td>
<td>Do.</td>
<td>Sand, clay</td>
<td>600</td>
<td>75</td>
<td>1,400</td>
</tr>
<tr>
<td>27</td>
<td>Do.</td>
<td>Do.</td>
<td>Sand, gravel, clay</td>
<td>650</td>
<td>80</td>
<td>2,000</td>
</tr>
<tr>
<td>28</td>
<td>Do.</td>
<td>Do.</td>
<td>Silt, sand, clay</td>
<td>900</td>
<td>43</td>
<td>1,700</td>
</tr>
<tr>
<td>29</td>
<td>Steel construction</td>
<td>Do.</td>
<td>Alluvium</td>
<td>370</td>
<td>45*</td>
<td>600</td>
</tr>
<tr>
<td>30</td>
<td>Do.</td>
<td>Do.</td>
<td>Do.</td>
<td>210</td>
<td>58</td>
<td>600</td>
</tr>
<tr>
<td>31</td>
<td>Do.</td>
<td>Do.</td>
<td>Do.</td>
<td>430</td>
<td>37*</td>
<td>740</td>
</tr>
<tr>
<td><strong>MOSTLY COHESIVE SOIL.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>Steel construction</td>
<td>Open excavation</td>
<td>Clay</td>
<td>450</td>
<td>75</td>
<td>1,500</td>
</tr>
<tr>
<td>33</td>
<td>Cast iron</td>
<td>Do.</td>
<td>Do.</td>
<td>500</td>
<td>60</td>
<td>125</td>
</tr>
<tr>
<td>34</td>
<td>Steel construction</td>
<td>Do.</td>
<td>Do.</td>
<td>700</td>
<td>65</td>
<td>1,300</td>
</tr>
<tr>
<td>35</td>
<td>Timber construction</td>
<td>Pneumatic</td>
<td>Transported chalk</td>
<td>490</td>
<td>17*</td>
<td>675</td>
</tr>
<tr>
<td>36</td>
<td>Brickwork</td>
<td>Open</td>
<td>Clay and peat</td>
<td>520</td>
<td>18*</td>
<td>630</td>
</tr>
<tr>
<td>37</td>
<td>Do.</td>
<td>Do.</td>
<td>Soft blue clay</td>
<td>855</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>38</td>
<td>Concrete</td>
<td>Do.</td>
<td>Very stiff red clay</td>
<td>450</td>
<td>—</td>
<td>14</td>
</tr>
<tr>
<td>39</td>
<td>Do.</td>
<td>Do.</td>
<td>Very stiff red clay</td>
<td>1,912</td>
<td>—</td>
<td>10,235</td>
</tr>
<tr>
<td>40</td>
<td>Do.</td>
<td>Do.</td>
<td>Very stiff red clay</td>
<td></td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

* Land caisson, depth below ground.
soil constituents but mostly to variation in the lubricating effect of excess water. In any case, after a short lapse of time the skin friction generally builds itself up to much greater values, as is well known in tests of loading and extracting piles in these soils, but at the same time the ultimate supporting value of the cylinder or caisson is likely to be determined by the ability of the sub-soil to support the vertical load upon it, consisting of the end bearing and the total of the skin friction. Thus it frequently happens that, if the end bearing is not alone sufficient to take the whole load, it is not possible to use fully the supporting value of the skin friction that ultimately develops.

It is sometimes feasible to assist the sinking of caissons by lubricating the outside by jets, for example when skin friction is to be disregarded in determining the safe load of the caisson, the bearing stratum at the level of the cutting edge being expected to take the whole load; but the use of lubricating jets is limited by their practical effectiveness over the large areas usually involved. A disadvantage in the use of jets to assist sinking is the erratic sinking that may follow, with marked tendencies for the caisson to sink out of the vertical. Table XV, mostly obtained from data collected by H. L. Wiley,\(^{(9,4)}\) supplemented by the writer, gives examples of recorded skin friction in sinking caissons arranged as far as possible according to the types of soil penetrated.

An example of the skin friction sinking steel cylinders about 33 ft. into river mud is the jetty for the Ford works at Dagenham where it was reported\(^{(9,6)}\) that the sinking effort for 12-ft. diameter cylinders was about 75 tons and for 14-ft. 6-in. diameter cylinders 85 tons. The average sinking rate was quoted by Sir Henry Japp as 142 ft. and 101 ft. per week respectively. On meeting a layer of cemented gravel, however, the load to cause further sinking exceeded 350 tons. To remove the cylinders an uplift of only 1-2 cwt. per square foot was needed to start them, and less while rising.

**Working in Compressed-air.**

The recent tendency to avoid working in compressed air in civil engineering is no doubt due to the availability of new methods of construction such as the sand-island method. There are, however, many types of construction where compressed-air working must be used in order to be able to carry out the work at all, or to carry it out properly, or to carry it out at a reasonable cost. Examples are the excavation for caissons in soils with boulders and excavation for tunnels in water-bearing soils.

The air pressure equivalent to the hydrostatic head of fresh water is 0.434 lb. per square inch per foot of depth below the free water surface, and, since considerations of health limit to about 50 lb. per square inch the pressure in which men may work, the maximum depth below the surface usual for compressed-air working is about 115 ft. The pressure which it is necessary to maintain to prevent the inward flow of soil may be less than the equivalent hydrostatic head, but to keep the working chamber reasonably dry a slight excess is often maintained. To assist sinking, the pressure in the working chamber is sometimes reduced for short periods, usually during meal stops, when the soil and working conditions permit, and similarly sinking may be retarded by keeping up the pressure, both effects being more likely to be obtained with large and shallow caissons than with deep caissons small in plan area.
When the penetration is some way into impermeable soil such as clay, and an effective seal has been obtained by the clay against the sides of the caisson, it is sometimes possible to dispense altogether with compressed air and work at atmospheric pressure. This has been done in the case of caissons for bridge piers in Denmark and the conditions are then somewhat similar to the excavation for underground railway tunnels in the London blue clay. If the clay is not very stiff, however, there will then be tendencies for slow plastic flow of the exposed faces, including perhaps a gradual rising of the floor. Where the air pressure does not need to exceed about 18 lb. per square inch, that is, depths below water surface of less than about 45 ft., men may work, subject to local regulations, most or all the full day in the working chamber and only simple precautions are necessary to ensure their health.

Regulations and practice vary in different countries and also between one state and another in the U.S.A. The requirements in New York State are given in Table XVI, and are more severe than in most other states. In accordance with recent regulations in Great Britain (9.6) there must be an interval of at least five hours before a man may again be subjected to compressed air.

Table XVI.—Maximum Hours under Pressure During Any 24-Hour Period.

<table>
<thead>
<tr>
<th>Working pressure (lb. per square inch)</th>
<th>Probable desirable maximum †</th>
<th>New York State regulations*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 18</td>
<td>4 + 4 = 8</td>
<td>8 hours with ½ hour (min.) rest interval</td>
</tr>
<tr>
<td>19 to 26</td>
<td>3 ½ + 3 ½ = 7</td>
<td>3 hours plus 3 hours after 1 hour rest</td>
</tr>
<tr>
<td>27 to 33</td>
<td>3 + 3 = 6</td>
<td>2 hours ‡</td>
</tr>
<tr>
<td>34 to 38</td>
<td>2 ½ + 2 ½ = 5</td>
<td>1½ ‡ ‡ ‡ ‡ 1½ ‡ ‡ 3 ‡ ‡</td>
</tr>
<tr>
<td>39 to 43</td>
<td>2 + 2 = 4</td>
<td>1 hour ‡ ‡ 1 hour ‡ ‡ 4 ‡ ‡</td>
</tr>
<tr>
<td>44 to 48</td>
<td>‡ ‡ ‡ ‡ ‡ ‡ ‡ ‡ ‡ ‡ 6 ‡ ‡</td>
<td></td>
</tr>
<tr>
<td>49 to 50</td>
<td>‡ ‡ ‡ ‡ ‡ ‡ ‡ ‡ ‡ ‡ 6 ‡ ‡ ‡</td>
<td></td>
</tr>
</tbody>
</table>

* Typical of several Eastern States in America. These limitations, coupled with unusually high rates of pay, make for the avoidance of compressed-air work in the areas concerned.
† For men medically examined, not overweight, not over 40 years of age.
‡ For men medically examined, not overweight, not over 45 years of age.

The quantity of free air to be supplied may be calculated from the CO₂ content of the open air and that adopted as the maximum for the working chamber (this is not in any way related to compressed-air sickness). Considerable increase in the capacity of the compressor over the calculated requirements will usually be necessary to cover leakage. By the method of Dr. Haldane the cubic feet of free air required per man-hour is

80

Permitted percentage increase in CO₂, or for example, if the open air has a CO₂ content of 0.04 per cent. and the maximum percentage in the working chamber is taken as 0.10 per cent. (a value often used and conservative for compressed-air work), the free air required per hour per man is 1333 cu. ft. More recently (9.6) the air supply is required to be 600 cu. ft. per hour per man measured at the pressure in the working chamber.

As any failure of the air pressure system results in serious risk of fatal accident to the men through release of soil and water kept out of the working chamber by air pressure, in addition to careful anchorage of the air lock it is necessary
to provide against failure in the air supply. The air-pressure pipe should be taken down inside the men’s access shaft and be everywhere out of the way of accidental damage but visible for inspection. Owing to the air pressure in the working chamber, compressed-air tools lose efficiency because of the back pressure and tend also to build up the pressure in the working chamber, so that it is only in shallow depths that they can be used successfully.

Explosives are sometimes used for breaking up soil to facilitate excavation, the firing being done during meal stops, or some means provided to expedite the ventilation so as to clear the fumes promptly to enable the men to re-enter. Care is desirable not to place shots near to the cutting edge in order to prevent damage to it, and also when it would result in subsequent increase in loss of air by leakage.

Carl Jansen (9-7), describing work in U.S.A., explained why compressed-air working is avoided so far as possible by drawing attention to the additional cost. For example, for work under air at a depth of 99 ft. to 110 ft. below water in accordance with current practice in the U.S.A., only one hour of actual work is done for the pay of an eight-hours’ day.

Caisson Sickness (or "Bends").

After the first few minutes, working in compressed air is only slightly different from working at normal pressure, but generally the high temperature of the air is noticeable and there is a feeling of high humidity by reason of the lack of evaporation of perspiration. Apart from minor phenomena such as sharp pains in the ears (usually relieved by blowing the nose once or twice), inability to whistle, and the ability to make greater physical effort before the breathing is affected, the conditions are similar to working at normal pressure. It is on leaving and afterwards that the effects may be of importance.

Forms of ill-health to which caisson sinkers are liable after working under compressed air have been covered in the past by various writers, of whom it may be sufficient to refer the reader to von Schröter, (9,8) Sir Leonard Hill, (9,9) and Dr. F. L. Keays, (9,10) who have all given extensive data obtained from the carrying out of large contracts and from experimental work, particularly Dr. Keays in connection with the East River tunnels at New York. The work of the late Prof. J. S. Haldane (9,11) should be mentioned particularly for the medical aspects of deep diving. There has also been more recent medical research. (9,12), (9,13)

Caisson sickness may be roughly divided into the more or less harmless transitory consequences of compressed-air working, and the very serious effects that may arise from too rapid decompression. All consequences are, however, caused by the drop in the pressure on leaving the caisson liberating air in the system, of which only the oxygen is readily absorbed by the blood, leaving the nitrogen either to be expelled or to form bubbles and impede the circulation, muscular movements, or the senses. Exercise during decompression is no longer recommended.

An explanation of the cause of "bends" given by Sir Henry Japp (9,15) is as follows: A bottle of aerated water is taken as an analogy to the fluids of the human system. If such a bottle is held on an angle to provide a larger surface of water, and the cork is gently eased from the neck, only a slight escape of small bubbles is observed; but if the bottle is held upright and the cork is suddenly removed, the effervescence is so great that the liquid froths out at the neck of the
bottle. In the same way after a man has been immersed in air pressure his blood and tissues become charged with gas. If he decompresses slowly the gas does not form into free bubbles, but gradually escapes from the lung surface without harm; on the other hand, if decompression takes place too quickly, free bubbles are formed which may lodge in the heart, the brain, and the spinal column, and as they enlarge with the addition of other bubbles and their natural expansion, they may cause a serious rupture resulting in paralysis or death, or these bubbles may cause a froth in the blood and stop the circulation.

Careful observance of the appropriate slow rate of decompression is generally sufficient to prevent both the occurrence of local sharp pains (generally in the knee-joints if not massaged) and the likelihood of any more serious consequences. Any ill effects occur only after decompression, usually within 15 or 30 minutes after leaving the air lock, and practically never later than six hours afterwards. The liberation of nitrogen in the system shows itself often in harmless prickling sensations on the skin, sometimes taken as an indication that no more serious effects are likely to occur. The appearance of a mottled skin, however, is a sign of danger. Owing to the greater absorption of gas in the system with fat men than with thin men, the latter are less susceptible to ill effects from compressed air working.

**Decompression.**

Decompression may be carried out by gradually lowering the air pressure in the air-lock at a rate not exceeding that desirable, or it may be done more rapidly. In the latter case, the workman is decanted (transferred) to a recompression chamber of the type shown in Fig. 132 where the pressure is about equal to the working pressure, and he is then “decompressed” at the desirable slow rate similar to the procedure adopted when the workman is leaving the caisson through the air lock. The medical lock needs to be well heated to overcome the chilling effect of decompression and the same applies to the air locks on the caisson. Fig. 133 is an external view of the medical lock used by divers inspecting foundations inside the caissons for the San Francisco-Oakland Bay bridge. 

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**Fig. 133.—A Medical Lock.**
Various methods and rates of decompression have been used in the past, the rate of decompression in particular still differing widely. As the amount of the liberated nitrogen is to some extent dependent upon the duration of the exposure to pressure, working time may have to be considerably reduced, for health reasons, for pressures in excess of about 30 lb. per square inch. Thus, on one contract in the past, where the working pressure was 48 lb., the fatal cases were stopped when four-hour shifts were reduced to one hour. In addition, slower rates of decompression become necessary for a longer exposure to any given pressure. Prof. Haldane's method, which has been used by naval divers, is to reduce the pressure rapidly from that of the working chamber to half the absolute pressure; thus for a 30-lb. working pressure the decompression would be to 8 lb. in the first three minutes, and subsequently reduced at a constant rate of from six to nine minutes for each remaining pound of pressure according to the duration of exposure. Prof. Haldane showed that there are practically no risks in decompression from 18 lb. gauge pressure. While the rates of decompression by some rules lead to a very long total time for decompression, Sir Henry Japp's\(^{(9,15)}\) used successfully a rate definitely higher than the present New York State regulations when the times of exposure are taken into account. Table XVII gives both rules for comparison.

### Table XVII.

<table>
<thead>
<tr>
<th>Gauge pressure (lb. per sq. in.)</th>
<th>Sir Henry Japp's rule</th>
<th>New York State regulations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reduce pressure in three minutes to (lb.)</td>
<td>Total time in air lock after 8 hours' work (minutes)</td>
</tr>
<tr>
<td>27</td>
<td>6</td>
<td>9</td>
</tr>
<tr>
<td>30</td>
<td>7½</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td><strong>After 3 hours' work</strong></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>8½</td>
<td>25</td>
</tr>
<tr>
<td>35</td>
<td>10</td>
<td>35</td>
</tr>
<tr>
<td>40</td>
<td>12½</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td><strong>After 2 hours' work</strong></td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>13½</td>
<td>37</td>
</tr>
<tr>
<td>45</td>
<td>15</td>
<td>42</td>
</tr>
<tr>
<td>50</td>
<td>17½</td>
<td>48</td>
</tr>
</tbody>
</table>

It is preferable for the rapid first stage of decompression to reduce to half the absolute pressure (as in Prof. Haldane's and Sir Henry Japp's rules) rather than to half the gauge pressure as in the New York rule. Whatever total time for decompression is adopted the decompression curve should preferably take the form of reducing the decompression rate more and more slowly as the atmospheric pressure is approached.

The British regulations for work in compressed air\(^{(9,16)}\) specify the degree of supervision, minimum ventilation, size and operation of man-locks, provision of attendant, and fastest permissible rates of decompression. The minimum supply of fresh air is 10 cu. ft. per minute per person. The wet-bulb temperature
of the working chamber may not exceed 80 deg. F. A medical lock must be provided when the working pressure exceeds 18 lb. per square inch; for this or greater pressure each worker is required to wear a label for guidance to others should he be taken ill after leaving work. Hot drinks must be available when leaving the man-lock and when at the medical lock.

Table XVIII gives the maximum rates of decompression.

**Table XVIII.**

<table>
<thead>
<tr>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>&quot;Basic pressure&quot;</strong></td>
<td><strong>Lowest permissible pressure in first two min. after starting decompression (lb. per square inch)</strong></td>
<td>Fastest permissible reduction of pressure from figure in Section 2 to zero. Shortest permissible times (T) in minutes, and fastest permissible rates (R) in minutes per lb. for the working periods in the different columns.</td>
</tr>
<tr>
<td><strong>&quot;Working period&quot;: More than (c) but not more than (d) hours</strong></td>
<td><strong>(c) (d)</strong></td>
<td><strong>(c) (d)</strong></td>
</tr>
<tr>
<td></td>
<td><strong>31-4</strong></td>
<td><strong>21-3</strong></td>
</tr>
<tr>
<td></td>
<td><strong>T Min.</strong></td>
<td><strong>R Min. per lb.</strong></td>
</tr>
<tr>
<td>18</td>
<td>20</td>
<td>2</td>
</tr>
<tr>
<td>24</td>
<td>26</td>
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</tr>
<tr>
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<td>14</td>
</tr>
<tr>
<td>48</td>
<td>50</td>
<td>17</td>
</tr>
</tbody>
</table>

**Diving.**

Inspection and work are often needed to be done by divers, for example the examination of the completeness of an excavation under water inside cylinders, and to burn off sheet-piling under water. This work is usually done by specialist firms and the reader is referred to draft regulations (9.16) or books (9.17) on the subject should be consulted.

Note.—Bibliographical references for Chapter IX are given on page 259.
CHAPTER X
CONSTRUCTION OF CYLINDERS AND OPEN CAISSONS

Open Wells.

The Chicago method,\(^{10-1}\) which was first used in Chicago about 1892, is suitable for providing foundations for heavy buildings and other structures when soil of adequate supporting value is situated 70 ft. or more below the surface, and the intervening strata are mainly impervious and cohesive. Essentially it is an open well method, and only differs in details of execution from a similar method that has been used in Paris and in other countries.

Methods which provide a cylindrical concrete pier are usually of themselves more costly than the alternative of piling if the depth is less than about 50 ft., but if the soil is generally impervious and the loads are heavy, so that a single pier avoids large pile caps under the supported structure, open well methods are to be considered. One advantage in built-up areas is the avoidance of pile driving, but the method is not efficient where the loads on the piers are much limited by the ability of the bearing stratum to support heavy local loading.

![Diagram of the Chicago Open-well Method](image)

Fig. 134.—The Chicago Open-well Method.

The procedure in Chicago (where the sub-soil is clay), Detroit, and other places where the method is used, has been to excavate by hand in 5-ft. stages and line the circular side with timber as shown in Figs. 134 and 135 before proceeding to the next stage. It is not essential to taper the lagging as shown, and with reasonably stiff clay successive stages of lagging may be butted so that the rings are all the same diameter. After the hardpan, or other good bearing stratum, is reached, the hole is filled with concrete, usually of a rather weak mix, and sometimes the steel rings and the staves are recovered during concreting. The soil is soft clay, and a tight fit is usually ensured by the timbering, so that with due precautions the method then does not cause any sub-soil disturbance.

When water-bearing strata are encountered, steel sheet piles have been driven in place of timber to form the lining for the lower part of the well. The sheet piles are interlocked before commencing driving, and driven each in turn a short distance. The success of this method then depends on the sheet-piles

* References thus\(^{10-1}\) refer to the Bibliography on page 225.
obtaining a cut-off seal in the clay. Alternatively the timber lagging is sometimes also used inside the sheet-piling as the excavation is carried downward and the space between packed with puddled clay.

In the case of the 52-storey tower of the Cleveland Union Terminal,\(^{10,2}\) where the sub-soil consisted of a soft plastic clay to a depth of about 70 ft., the cylindrical open well method was also used after careful consideration of the possible alternatives. The bearing stratum consisted of hardpan, a hard compact clay mixed with gravel immediately overlaid by a water-bearing stratum containing rock and showing evidence of glacial action. The whole load of the tower was carried in this way from sixteen columns at rail level, by the concrete-filled wells. Where the bottoms of the piers rested on clay they were belted out at the base with surfaces sloping at one in two. The foundation loads vary from 3150 to 4450 short tons with diameters from 8 ft. 8 in. to 10 ft. 4 in., the allowable soil pressure beneath the bells being finally fixed at \(5\frac{1}{2}\) short tons per square foot, with an alternative limit of \(7\frac{1}{2}\) short tons per square foot when skin friction was ignored. Tenders were obtained for alternative designs for the sixteen foundations bearing on clay at an elevation of \(-83\)° and alternatively extending to bed-rock at an elevation of \(-155\)°, equivalent to a total depth of 204 ft. The latter method was adopted although more expensive, and the excavation was carried through the clay using tongued-and-grooved hardwood lagging similar to the Chicago method, neither lagging nor rings being recovered from the wells during concreting. During construction of the wells a general subsidence of the surface of several inches occurred over a restricted area, due, no doubt, to lateral flow of the deep clay strata into which the wells were being excavated, and perhaps to a small extent by plastic flow in filling the small clearance between the excavation and the setting of the lagging.

![Fig. 135.—Chicago Open-well Method.](image-url)
Cylinders.

Open cylinders are very convenient for forming the foundations of bridges over rivers, and are also used fairly extensively for foundations on land where the depth to a satisfactory bearing stratum, or the concentrated loads to be carried, makes them more economical than driving piles. The cylinder is a light shell that invariably becomes part of the permanent construction.

In water, if the penetration of the soil is small, the shell is sunk by its own weight without dewatering, the soil being excavated by grab or by water jet and pumping until the bottom edge reaches soil of adequate supporting value. On land, or if the penetration is greater, kentledge may be necessary.

When this method is used for bridge piers, most frequently two cylinders are sunk a slight distance apart with bracing between (Fig. 136), while, for the pivot piers of swing bridges, six or more cylinders are sunk around the periphery of the bearing circle of the swing span and braced together at the top between themselves and to a similar, usually larger, cylinder at the centre. The cylinders may be of cast iron, mild steel, or concrete, steel being generally most suitable with this method for small diameters, and reinforced concrete for large diameters.

![Diagram](image)

**Fig. 136.—Typical Arrangement of Cylinders for Bridge Piers.**

of the bearing circle of the swing span and braced together at the top between themselves and to a similar, usually larger, cylinder at the centre. The cylinders may be of cast iron, mild steel, or concrete, steel being generally most suitable with this method for small diameters, and reinforced concrete for large diameters.

Open Caissons.

The diameter of hollow cylinders of this type may be determined more by the method of excavation than the size required for the load to be supported.

Open caissons of reinforced concrete may have the first section concreted in a pit in the ground, and the concreting proceeds in lifts as the cylinder is sunk by excavating inside, the excavation being sometimes several feet in advance of the sinking of the cylinder. Guide piles may be necessary to ensure that the sinking is plumb for the first 20 ft. or so. If the soil excavated is dry, bucket skips may be filled by hand, but it is worth while drawing attention to the danger of men working in the well when the cylinder encounters water-bearing fine sand which may become quick and therefore lose supporting value due to upward seepage. There is the additional risk that if the sinking of the caisson does not almost immediately cut off the flowing sand material will be drawn through from the adjoining area with the possibility of settlement of adjacent structures.
If considerable resistance to sinking develops, keelledge consisting of concrete blocks or other heavy materials may be necessary to continue penetration. On occasions water jets have been used to reduce the friction outside the shell.\(^{10,3}\)

When the caisson reaches rock which is overlaid with clay no difficulty is usually experienced in cleaning out in the open, but if permeable strata overy the rock an air lock is bolted to the top of the caisson and the cleaning-out done under compressed air.

The use of rotary excavators is likely in the future to become more general for open caissons of small diameter and to be particularly suitable where the soil is clay. By one method a steel cylindrical shell is provided with hardened steel teeth and the shell is rotated and penetrates while the cylinder is kept full of water, using water jets acting along the cutting edge. The water jets are arranged to give a helical flow to the rising water, and the skin friction is reduced to a great extent by the water flow coming up outside to the surface. This permits of cutting through boulders and to some extent through rock. Fine material is partially removed during sinking by the rising water. The rest is excavated subsequently after the top of the cylinder is taken off. By another method, usable when the soil penetrated does not contain boulders or particularly hard soil or shale, the rotated shaft is fitted with scarifying tools extending to the diameter of the cylinder being sunk, transforming the soil into mud and so enabling a steel shell to be driven to the bearing stratum, after which the enclosed mud is excavated and the shell filled with concrete.

**Steel-cylinder Foundations.**

Where a water-bearing stratum is encountered, steel cylinders may be used in sections, or strakes, connected together and jacked down as the excavation

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Fig. 137.—Steel Cylinders used for the Foundations of the Manhattan Building, New York.
proceeds. A well-known example of this is the Manhattan Company building \(^{(10.4)}\) in New York City (Fig. 137). This method is suitable where the foundations of a new building are to be constructed before the existing building is demolished, since the jacking down can be done against the superstructure of the existing building. The same applies when underpinning an existing structure.

Apart from the use of steel cylinders for bridge piers, they are also sometimes used as piers for wharves. The procedure is the same as for bridge piers, that is to say they may themselves be sunk to hard bearing strata, or they may be sunk to beyond the expected level of scour and piles driven inside the enclosed area down to satisfactory support. (See also page 195.)

![Fig. 138.—Welded Steel Cylinders up to 107 ft. long delivered in one piece for Driving by Impact.](image)

**Impact-driven Cylinders.**

While in the preceding examples the cylinders are sunk by excavating or pumping from the inside with the assistance of the self-weight of the cylinder—sometimes with the help of kentledge and sometimes, in the case of underpinning, by jacking down—with the smaller sizes of cylinders, if the shell is made strong enough, they may be driven by impact, using, say, a large size steam hammer and a special driving head or helmet. An instance of this is the recent construction of the wharf for a ship repair dock at New Orleans,\(^{(10.5)}\) in which the cylinders were 4 ft. to 4 ft. 6 in. in diameter and from 77 ft. to 107 ft. in length. The cylinders (Fig. 138) were made and delivered each in one piece, the shell being \(\frac{3}{4}\) in. thick at the top and \(\frac{3}{8}\) in. thick at the bottom. It was expected that the usual method of sinking cylinders and excavating under water from the inside would have been likely to involve overcoming a skin friction as high as 1100 lb. per square foot of the embedded surface area, and, in view of the
small diameter, it was doubtful whether mechanical excavation inside the shells would have been feasible. A special driving head was designed with grooves to suit the various diameters and the grooves so placed as to give contact against the inner edges of the cylinders since it was to be expected that the top edges would better withstand being spread outwards than inwards during driving. The leaders of the frame were provided with sufficient spread to take the largest cylinder. The driving was assisted by a powerful jet pump capable of developing a pressure up to 1000 lb. per square inch with a comparatively small volume of water, or a delivery up to approximately 800 gallons per minute under 300-lb. pressure, and it was found that the jet was powerful enough to keep the earth practically fluid.

The longitudinal and circumferential joints in the steel plates were machine welded and no cover plates were used. The cutting edge, however, was reinforced by the addition of a 24-in. by \( \frac{1}{4} \)-in. plate. It was found that some of the handmade welds failed under the driving conditions experienced, but that good welding done by machine at the fabricator's works withstood any amount of hard driving. Some difficulties were expected and encountered due to sub-surface obstructions, but the jet was powerful enough to cut through timber by the force of the water alone.

The cylinder could not be excavated closer than about 8 ft. from the bottom edge without danger of a blow-in, and in trying to excavate deeper one cylinder suddenly sank 4 ft. under its own weight. The excavation was carried out by air compressor with agitation from an ordinary jet pump through a 6-in. diameter pipe with a \( \frac{1}{4} \)-in. air connection turned up into the bottom. An auxiliary pump kept the water inside the cylinder at any desired level. The concrete was placed under water by means of a concrete pump with the delivery pipe run down inside the cylinder to the bottom, a plug in the end of the pipe keeping the water out until the pipe was filled with concrete. In this way the seal is not lost during concreting.

Impact-driven reinforced concrete cylinders 3 ft. in diameter in lengths up to 145 ft. with both ends open were used in large numbers for the foundations for the Lidingö bridge, near Stockholm,\(^{10,6}\) being driven through 60 ft. of water and another 50 ft. (about) of soft clay by a 10-ton hammer, using a special annular driving head. After driving, the tubes were cleared of clay by the use of a mechanical churning device and the mud ejected by pumping.

**Cylinders and Piles.**

For bridge piers and the like in water which is under-laid with a soft soil suitable for driving piles, the cylinders may be placed in the position to be occupied by the pier and maintained in position by guide ropes, or may be secured in position by a temporary piled staging upon which is mounted a derrick or pile frame for driving the piles through the bottom of the cylinder. The cylinder is then sunk under its own weight, not necessarily farther into the soil than is required to protect the piles, as the pier load is taken by the concrete filling which also forms the pile cap. Since the cylinder in this case becomes merely the formwork for the concrete filling it is usually of light construction of steel or reinforced concrete. In this event the cylinder shell may be designed for the ring tension resulting from the lateral pressure of the concrete filling and
to resist handling stresses and accidental impacts. External water pressure is only of importance for the design of the shell if the cylinder is likely to be pumped out during sinking, and this is not likely to be feasible in this case.

Generally speaking, for steel cylinders a thickness of shell of less than \( \frac{1}{2} \) in. is unusual, and, if the cylinder is not likely to penetrate into the soil sufficiently to prevent the possibility of a blow-in, the water level inside must be kept high, by pumping back if necessary, and the concrete seal will need to be placed under water.

The piles driven down inside the cylinders may be driven under water by a drop hammer by using a follower and hanging leaders, as mentioned on page 47 and reinforced concrete piles are generally used. In this case the diameter of the cylinders is determined by the minimum efficient separation of the piles rather than by the maximum compressive stress in the concrete filling of the cylinder considered as a column.

Opinions differ on the minimum spacing apart of the piles, but, without serious loss of effective supporting value, the pile heads should be spaced apart centre to centre not nearer together than \( 4b \) where \( b \) is the breadth or size in plan of the piles being used, the piles being driven to a slight rake if necessary to spread the area on which the several pile shoes bear. Even so, the supporting value of a cluster of piles like this bearing on clay will be very much less than the sum of the driving resistances of the individual piles as indicated by a good impact formula. (See page 10.)

Sheet-pile Cylinders.

An alternative to sinking cylinders is to drive sheet-piles in a circle and use them as formwork for concrete filling after the enclosed space has been excavated. The diameter of circle formed is, however, limited by the available play in the interlocks—which, with most European sections of sheet-piling, does not permit the driving of small circles unless the piles are bent.

Using American sections, to form circles of less than about 14 ft. diameter, half the piles are bent with the interlock fingers in and half with the fingers out, as in Fig. 139 (a). In this way the minimum clear diameters of cylindrical wells formed by sheet-piles can be reduced, so that, with the 10 deg. swing in the interlock also assisting, diameters of 5 ft. to 7 ft. are possible (Fig. 140), for flat sections such as Carnegie M107 and Bethlehem SP6, while very small clear inside space, down even to 2 ft., can be obtained using four piles only bent to form 90 deg. corners.

The sheet-piles must be pitched and interlocked together around a template and driven each in turn a small distance, otherwise closure of the circle is impossible, or at best there is some distortion of the piles or shape of well. In the case of arched American sheet-pile sections it is possible, with sacrifice of interlock strength, to drive piles with the interlock reversed so that all the arches project on the one side relative to the interlocks, as in Fig. 139 (b). This is sometimes useful in cases of limited space or to save concrete when working to a given line. With one combination of European sections, viz. Larssen types 2 and 10A used alternately, the same result is obtained, and is possible because the interlock on the type 10A is of opposite hand to the interlocks of the other sections of this make.
Fig. 139.

Fig. 140.—Steel Sheet-piles driven to form a Cylinder.
With European section sheet-piles, cylindrical wells are seldom used, but, although the interlocks do not have any appreciable angular play, circles can be formed by bending all the piles. The more common method is to form a rectangle or a square in plan, using either bent or fabricated corner piles, for example in the case of Larsen sections as in Fig. 139 (c).

For small exposed heights the use of internal framing may be omitted, but this is limited by the possibility of distortion by earth pressure of the sides of the rectangle, and therefore subsequent extraction of the piles should not be reckoned upon. For larger exposed heights the well must be large enough in plan for the crane skip or a grab to pass freely between the walings, and this results in the minimum rectangular sheet-pile well being about 8 ft. square. The exact dimension will be decided by the multiples of the net width of the type of sheet-pile used and the type of the corner piles, the type of corner pile being decided by the number of piles in the side being odd or even as will be seen from Fig. 139 (c).

Prestressed Concrete Cylinders.

A recent example of the use of prestressed concrete cylinders is a jetty on the Thames at Erith (10–44) (Figs. 141, 142 and 143) for ocean-going vessels of 14,000 tons displacement with a draft of about 28 ft. The jetty is L-shaped in plan with a straight approach-arm comprising eight bays each of 50-ft. span. The jetty head comprises twenty-two bays of 25-ft. span each. In the interests of
Fig. 142.
DETAIL OF CUTTING RING.

INNER PLATE 1/2" M.S.
1"THICK DIAPHRAGMS

DETAIL OF BAR COUPLING.

BAR NOT SHOWN FOR CLARITY.

CAVITY JOINT.

DETAIL OF 2'-6' EXTENSION TO CYLINDERS.
(3'-6" extension similar)

BEARING PLATE
2'-6' SPIRAL 1'-PITCH 6'-LONG.

JOINT BETWEEN UNITS.

Bearing plate, M.S. Tube, M.S.
THINNEN 16G. THICK.

BAR not shown for clarity.

Fig. 143.
speed and economy, prestressed precast concrete construction was adopted for the main structural members combined with a cast-insitu concrete deck slab. The foundations consist of seventy-two prestressed concrete cylinders. In the jetty head, all the cylinders are 65 ft. long; those in the approach arm vary in length from 35 ft. and 65 ft. Each cylinder is built up of a number of precast concrete rings of 6-ft. external diameter and about 5 ft. long; the thickness of the wall is 6\(\frac{1}{4}\) in. The cylinders were assembled vertically in a special berth on-shore. After assembly, the units were connected by post-tensioning sixteen 1\(\frac{1}{4}\)-in. diameter Macalloy bars spaced symmetrically. The bars were inserted through 2-in. diameter cored holes and, after stressing, grouted with colloidal grout. The same prestress was applied to all the cylinders so that the spacings of the bars would be equal and assembly and sinking operations could be standardised.

To facilitate sinking, each cylinder was fitted with a cast-steel shoe; the end nuts on the prestressing bars were located within the ring of this shoe which was filled with mortar prior to launching. The weight of a complete 65-ft. hollow cylinder was about 40 tons. The cylinders remained vertical while being floated into position, pitched, and sunk. They are founded in hard white chalk in a matrix of soft chalk and flints, overlaid with a thin layer of sand and chalk and a layer of flint gravel. The open-ended cylinders were sunk by grabbing in combination with loading with kentledge blocks. When a cylinder had been sunk to its final depth, silt and mud were removed by a diver preparatory to the placing of a plug of concrete. To achieve a good bond between the precast concrete and the cast-insitu plug, the inner surfaces of the topmost two and bottom three cylinders were roughened. The first section of the plug was formed by placing gravel under water in the bottom of the cylinder, about 20 ft. deep, and injecting colloidal cement into the bottom of the gravel. The rising grout displaced the water and filled the interstices to form a solid concrete plug. The cylinder was then pumped dry and filled with lean concrete.

Open Caissons.

With the open caisson method weight can be added, usually by building up the caisson to assist sinking as the excavation proceeds by grab through the open shafts or by hand using crane skips when the penetration has reached a safe depth giving a definite cut-off in impermeable soil.

A circular shape in plan is usual for the smaller sizes because it is most economical in material, but for larger piers, where in any case internal bracing or framework is necessary, the shape of the open caisson will often be decided by a combination of the need not to restrict the waterway and to suit the bearings required for the bridge girders. Because of this a double D shape in plan is most usual for bridge piers and in special cases, for example, where ice floes are encountered, or in navigable waterways where vessels may collide with the pier, it may be provided additionally with a sharp edge along the upstream face or the section in plan otherwise modified at the water line and below it.

If open caissons reach rock at, or close to, the bed of the waterway, they may be sealed by placing around the outside a layer of concrete, provided the concrete is in effective contact with the rock, any overlaying deposit having been first removed by water jet or by diver.
Typical examples of open caissons for bridge foundations are shown in Fig. I44, and Fig. I45 gives the general dimensions of the three bridges concerned. Fig. I46 shows the soil penetrated in each case. For the bridges at Knysna and Umkomaas rectangular open caissons were used, but although that at Umbogintwini is described as a cylinder pier it is correctly also an open caisson. Both the caissons and the cylinders were built to heights of 6 ft. to 12 ft., either on artificial islands or in excavations before sinking was commenced. No guides were used during the sinking of either the caissons or the cylinders, and it is stated that this was fully justified by the results.

At Knysna the penetration was made difficult by tree stumps and boulders, although the stiff mud acted as a cut-off and made excavation possible in the dry. Sand blows were frequent, and one filled the pier caisson to a depth of 9 ft. After springs broke through, the running silt added greatly to the quantity to be excavated. Sinking was more or less gradual but it was sometimes necessary to excavate to a depth of 4 ft. below the level of the kerb before movement took place. When the abutment caisson was down about 24 ft., and nine days' excavation had had no effect, the pumps were started, and after lowering the water to within about 13 ft. of the cutting edge the caisson plunged 2 ft. 9 in. Before finally cleaning off the bottom, bags of concrete were emptied and placed by divers between the hard bottom and the cutting edge, thus closing off infiltration of silt. The sealing layer was 1 : 2 : 4 concrete. The caisson was subsequently filled with sand.

The following extracts from the same paper describe the sinking operations at Umkomaas and Umbogintwini and are of interest particularly to show examples of site difficulties.

"At Umkomaas, sinking was less difficult, No. 2 pier being sunk almost entirely by grab. As a precaution against the caisson drifting or getting out of plumb the grab was first worked in the middle well for an hour or two and then in the end wells alternately, sounding tests being made with a view to preventing the depth of excavation in one end well materially exceeding that in the other. No. 1 pier was more troublesome, the caisson having to pass through a thick stratum of tough clay. When grabbing proved ineffective, pumps were set to work. A 3-in. centrifugal pump failed to empty the caisson, so it was supplemented by a 4-in. centrifugal pump, both being run by the engine of a motor-car which was fixed on top of the caisson. Two pulleys were clamped to the back wheels, and a pump was run from each. These were successful in dewatering the caisson, and 6½ days' excavation by hand caused a sinking of 2 ft. 6 in. The depth of the excavation was 4 ft. to 5 ft. below the cutting edge when the 3-in. pump broke down and grabbing was restarted. Poor results followed and, the defective pump having been put in order, both were set going again. When water had been lowered to within about 2 ft. of the cutting edge, the caisson plunged 4 ft. Only a very small blow took place, as the stratum being passed through was very stiff. The height of the caisson was increased to 24 ft., and the grab was taken into use again. Very little sinking occurred during the next seven days, although the excavation in the middle of the wells was 7 ft. to 8 ft. below the cutting edge. During the following night, however, the caisson sank about 3 ft. On this occasion the excavation was through the clay and into sand which, after the plunge, had refilled the caisson to 3 ft. to 4 ft. above the cutting edge. The next nine days' grabbing caused a gradual sinking of about 4 ft. Another bed of tough clay was then encountered, and after penetrating it for about 1 ft. it was decided to stop further sinking. During sinking there was never much departure from a level keel in either caisson, but when No. 1 plunged 4 ft. a lengthwise difference of 14 in. occurred which was corrected during the next 4 ft. of sinking.
Fig. 145.—Outline Diagrams of Three Bridges with Caisson Foundations as shown in Fig. 144.
(The soil penetrated is shown in Fig. 146.)
**UMKOMAAS NO. 1 PIER (LEFT)**

<table>
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<tr>
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<th>Scale Dye</th>
<th>Method of sinking</th>
<th>Weight or Air and Influence of Grounds</th>
<th>Depth (ft.)</th>
</tr>
</thead>
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<td>Open Excavation</td>
<td>Natural Ground level</td>
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</tr>
<tr>
<td>Tough Clay</td>
<td>0</td>
<td>Grab</td>
<td>HWL - 6'</td>
<td>6</td>
</tr>
<tr>
<td>Fine Lias Sand</td>
<td>0</td>
<td>Pumping only</td>
<td>Ceramic level - 20 Tons</td>
<td>12</td>
</tr>
<tr>
<td>Tough Clay</td>
<td>0</td>
<td>Grab</td>
<td>HWL - 6'</td>
<td>6</td>
</tr>
<tr>
<td>UMBOGTINTWINI PIER</td>
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</tr>
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</table>

**KNYSNA PIER**

Vertical Scale: 1 inch = 20 feet.

<table>
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<th>Strata</th>
<th>Scale Dye</th>
<th>Method of sinking</th>
<th>Weight or Air and Influence of Grounds</th>
<th>Depth (ft.)</th>
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</thead>
<tbody>
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<td>Diver</td>
<td>Cerber level</td>
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</tr>
<tr>
<td>Soft Mod.</td>
<td>0</td>
<td>Diver</td>
<td>Cerber level</td>
<td>20</td>
</tr>
<tr>
<td>Soft Mod.</td>
<td>0</td>
<td>Diver</td>
<td>Cerber level</td>
<td>30</td>
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<tr>
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<td>Diver</td>
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<td>0</td>
<td>Diver</td>
<td>Cerber level</td>
<td>100</td>
</tr>
</tbody>
</table>

**Fig. 146.—Subsoil Penetrated by Caissons shown in Fig. 144.**
"At Umbogintwini the cylinders gave little trouble. Concreting and sinking were carried out alternately in the two cylinders of a pier. The cutting edges were usually kept within about 2 ft. of one another, but on one occasion there was a difference of 5 ft. 6 in. and on another of 6 ft. 6 in. without causing any drift. When the tops of the cylinders were slightly above water level, temporary sections 4 ft. high and 4 in. thick, formed of poor concrete, were cast on top of the outer perimeter. As soon as sinking was stopped, a sealing layer of 1:3:6 concrete 4 ft. thick was placed through water by means of a skip. After seven days the cylinders were pumped dry and laitance was removed from the concrete surface. A core of 1:4:8 concrete was then deposited to within a foot of the top of the permanent cylinders.

Fig. 147.—Constitutional Sequence for Open Caissons arranged for Change to Pneumatic Method.

"As the slab had to be formed below water level, a close-sheeted cofferdam was next formed and the interior was excavated to the level of the bottom of the slab. It was then pumped out, the temporary rings were removed from the cylinders, and a 4-ft. thick slab was placed. The abutment and pier were subsequently built in mass concrete. The pier weighed about 205 tons and caused a settlement of 0.02 ft. The abutment cylinders were not checked for settlement. In both cases the cylinders were plumb on completion, and the greatest deviation from correct position was not more than 2 in. in any direction. Both with caissons and cylinders the contractors were very loath to incur expense in placing and removing heavy kentledge.

"At the left abutment a tough clay bed existed at 8 ft. below the river bed. Steel sheet pile cylinders 8 ft. in diameter were driven to a depth of 28 ft. below the river bed. They were then excavated, filled with 1:3:6 concrete, and capped similarly to the reinforced concrete cylinders."
Sometimes it is a desirable precaution with open caissons to construct them so that, if necessary, the excavation can be completed by the pneumatic method. A recent example of this is the south pier caissons of the First Narrows (Lion's Gate) suspension bridge at Vancouver,\textsuperscript{[10,7]} of which Fig. 147 shows the constructional sequence. The caissons were constructed on shore while the drilling and blasting of the site were carried out from floating equipment, the material loosened being removed by dredging. Each caisson was circular in plan with four steel trusses set in pairs at right angles enclosing a 10-ft. diameter steel shaft giving access for open excavation, or the pneumatic method should that be found necessary. Before being floated to position, the concrete substructure was surmounted by a timber cofferdam of 4-in. planking, 24 ft. high, further concrete placed and the cofferdam raised to give a total height of about 42 ft. before being placed in position. While it was first intended to seal the caisson to the rock by means of concrete placed by tremie on the outside in a depth of water of about 40 ft., it was found before excavation could start that there was leakage through a layer of sand between the solid rock and the tremie seal, but the pneumatic method was avoided by placing further concrete by tremie in this part after more carefully cleaning the bottom before pouring.

**Open-caisson Foundations on Land.**

Either the cofferdam method or piling is usually more economical than open caissons for foundations of moderate depth on land. If the foundation loads are moderately heavy and the depth to be penetrated is great, the cylinder method is worth considering. Open caissons are only used on land when the loads are heavy and the depth or other circumstances make piles or cofferdams unsuitable. An example of an open cast-insitu concrete caisson (\textsuperscript{10-6}) for foundations to carry a load of 1000 tons is shown in Fig. 148. The penetration was through layers of soft clay to fireclay at a depth of about 42 ft. Before the insitu foundation concrete was poured the excavation was belled out while the caisson was supported by heavy timber posts to prevent further downward movement.

**Floating Caissons.**

For large bridges it is sometimes more convenient to construct caissons in a dock or on a slipway and tow them to the position where they are to be sunk. This method may be adopted merely because it is more economical, but generally because it would be impractical to construct the caissons at the site of the pier, due, for example, to the depth of water and the difficulties of erecting a temporary pile staging.

An example of the latter was the foundations of the Tacoma Narrows bridge (\textsuperscript{10.9}) where the piers had to be placed in water that had a maximum depth of 200 ft. and had a current up to about 8 knots. The 2800-ft. central span enabled the piers to be placed in about 120 ft. of water, and the resulting total depths below the surface were 175 ft. and 224 ft. respectively for the west and east piers. The sub-soil consisted of glacial deposits of sand and gravel extending far beyond the elevations to which the caissons were sunk.

The caissons were constructed of reinforced concrete, 66 ft. in width and 120 ft. in length with external walls 3 ft. 3 in. thick, and subdivided into wells
approximately 13 ft. square by reinforced concrete cross walls 24 in. thick. To enable them to be floated, bottom doors were provided consisting of two layers of timber, one of 8 in. by 12 in. material and the other of 4 in. by 12 in., all seams being well caulked.

An important problem in this case was the anchorage of the caissons when floating and landing, and the method adopted of using reinforced concrete blocks of 600 short tons for this purpose is indicated in Fig. 149. These blocks were placed around each pier site, according to the expected requirements, in a circle of about 900 ft. diameter, the effectiveness of the blocks as anchorages being taken as their resistance to sliding at 0.4 of the weight after allowance for buoyancy and the vertical component of the cable pull. The anchor blocks were cast on barges and successfully dumped into water at pre-determined positions by admitting water into compartments along one side of the barges. The blocks were subsequently wrapped with cable by divers and proved by pulling tests to be satisfactory for forces 50 per cent. greater than required. The caissons were
CONSTRUCTION OF CYLINDERS AND OPEN CAISSONS

built up to a height of 36 ft. before towing to position and securing to the anchorages. They were then sunk by constructing additional lifts and adjusting the anchorage line continually during sinking and with changes in the tide. The penetrations of 55 ft. and 100 ft. respectively for the two piers were obtained

![Diagram of construction process]

Fig. 149.—Use of Concrete Blocks to Anchor Floating Caissons.

by excavating through the open shafts after removing the timber bottoms. A 25-ft. thickness of concrete was then placed by tremie pipes 12 in. in diameter. The piers during construction are shown in Fig. 150. This bridge may be remembered because of the failure of the superstructure a short time after completion, but the failure was connected with the unusually low width-to-span ratio of the deck and not in any way with the substructure piers or main towers.

Fig. 150.—Construction of Pier on Caisson shown in Fig. 149.
Caissons Floated to Position Inverted.

Sometimes there are several advantages to be obtained by constructing a caisson on a slipway upside down, and after towing to position inverting it by admitting water or adding ballast on one side. Fig. 151 shows an example of this as used in the case of the Little Belt bridge (10,10) in Denmark, and Fig. 152 shows the general dimensions of the bridge. In this case the caisson was built inverted because the upper horizontal deck enabled easier launching. As launched, the caisson had a height of 50 ft. to 60 ft. and a weight of about 7000 tons. The average time required for constructing a pier was reported to be as follows:

<table>
<thead>
<tr>
<th>Task</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Work on caisson before launching</td>
<td>5 months</td>
</tr>
<tr>
<td>Finishing caisson and placing it</td>
<td>6 &quot;</td>
</tr>
<tr>
<td>Making caisson ready for boring</td>
<td>3½ &quot;</td>
</tr>
<tr>
<td>Boring and sealing in pipes</td>
<td>2 &quot;</td>
</tr>
<tr>
<td>Work in chamber</td>
<td>1½ &quot;</td>
</tr>
<tr>
<td>Finishing pier and pier shaft</td>
<td>6 &quot;</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>24 &quot;</strong></td>
</tr>
</tbody>
</table>

Use of Sand Deposits for Sinking Caissons.

To avoid placing floating caissons upon very soft soil in the beds of waterways and the consequent risk of the caisson taking up a position out of the vertical, which it is difficult to correct, a sand deposit is frequently placed on the bed of the waterway and the caisson founded on this. It is then sunk through the sand and makes use of the lateral support of the sand during subsequent sinking. This method is, of course, not feasible if the velocity of flow of the water will shift the sand, in which case the cylinder or caisson must be sunk within a guiding temporary staging of timber piles or by the sand-island method subsequently described in which the sand is deposited within a retaining steel shell.

Advantage is sometimes taken of the ability of a sand deposit to distribute the load of a monolith or caisson over a wider bearing, thereby enabling monoliths or box caissons to be supported at a fairly shallow depth in circumstances where the sub-soil has extremely poor bearing value extending perhaps to great depths. A typical instance of this is the sub-soil in the waterway at Rotterdam, and the method that has been adopted in this and similar cases is to dredge away sufficient thickness of the silt sub-soil and replace it with a sand deposit so as to distribute the load of the new construction to a value which the silt or other soil of low bearing value is able safely to support.

The front cells of the caissons (X in Fig. 153) are sometimes left empty, so that the resultant force falls very close to the centre of the base of the caisson. The two further considerations then in the stability of the construction are the frictional resistance against sliding forward and the possibility of a sub-soil failure through over-loading.

Box Caissons.

In recent years box caissons have been used fairly extensively for quay walls where the site conditions enable a levelled sand or gravel bed to be prepared beforehand to receive them. If the channel bed is poor soil, which more
Fig 151.—Method of Launching Caissons in an Inverted Position and Overturning before Sinking.
Fig. 152.—General Dimensions of Bridge for which Caissons shown in Fig. 151 were used.

Fig. 153.—Method of Replacing an Inferior Subsoil to Found Box Caissons for a Quay Wall.
often than not is the case, the channel bed is dredged first and replaced by a sand raft on to which the box caissons may be sunk. For floating the caissons to the site it may be necessary to strengthen the side walls by internal strutting while they are being sunk into position, but, after sinking, the filling that is placed inside is determined by normal stability requirements, it being seldom an advantage, and often a disadvantage, for the filling to be concrete or for the weight of the filling to be heavier than is needed to obtain due stability against overturning, since the maximum pressure of the box caissons on the sand raft is at the forward edge along which the soil has least permanent ability to resist downward loading. Also, since any yielding of the subsoil below the sand raft, due to excessive subsoil stresses, would show itself by the caissons tilting forward, the maximum bearing is reduced by the bottoms of box caissons subject to lateral loading being usually provided with a projecting edge at the front and sometimes also at the back.

The use of this method for quays, and sometimes also for sea walls, has been favoured by continental engineers, particularly in the nearly tideless waters of the Baltic and the Mediterranean where other methods involving tidal working are not possible, but it has also been chosen in many cases where substantial tidal variation occurs, no doubt largely because of the economy it permits in those cases where (1) the depth is excessive for sheet-piling, anchorage for them is difficult or impossible, and the superimposed loading would add unduly to the modulus required; or (2) the subsoil cannot be relied upon to develop adequate passive resistance to forward movement, but, with the distribution of vertical loading by means of the sand deposit, the subsoil becomes suitable to take the additional vertical loading of the new construction.

"Sand-island" Method.

If conditions make stabilising the caissons difficult, the sand-island method may be suitable. The method is to sink on the river bed a large cylindrical steel shell, filling this with sand, constructing the caisson on top of the sand, and sinking the caisson through the sand and into the subsoil. This method was first used for the Suisin Bay bridge, and was subsequently used for the river piers of the New Orleans bridge. In the latter case the soil penetrated consisted of alternate layers of sand, silt, and clay, necessitating the foundations being carried to a depth of 170 ft. below water level and the sand island method enabled the use of the use of pneumatic caissons. The method used is shown in Fig. 154.

The method has the following advantages. The work can proceed without interruption through flooding of the river, as the shell is carried to above flood level; it is possible to control closely the sinking of the caisson for alignment; there is much better accessibility during construction of the caisson, since the work is carried out in the dry; the risk of a blow-in under the cutting edge during sinking is also reduced. Considerations of cost, however, would be unlikely to favour this method except in cases where the sinking of caissons directly into the soil could not successfully be done.

Before placing the steel shell a woven willow mattress measuring 250 ft. by 450 ft. was sunk over the area to prevent scour. After constructing a timber
staging around the position of the sand island, the shell, of 120 ft. diameter, was constructed in lifts of 10 ft. high, using \( \frac{1}{2} \)-in. steel plates with continuous angle flanges for joining the sections together. A height of 30 ft. was assembled on needle beams spanning falsework, and after lowering by means of twelve hoist frames (Fig. 156) the weight of the shell was again transferred to the needles and another section assembled above. The shell was thus supported from the top until it extended down to mattress level. The mattress was cut through by driving a sharpened steel pile round the inside of the shell, and the circular area of mattress then removed.

After filling the shell with sand, the construction of the caisson was commenced and it was sunk under its own weight by dredging inside the wells. After sinking had proceeded sufficiently, a timber cofferdam for the lower part of the pier was built and followed down into the sand with the caisson. The concrete sill to the caisson was then placed by tremie, and the base of the pier constructed

![Diagram](image)

**Fig. 154.**—The “Sand-island” Method of Sinking Caissons.

on top of the caisson. There were some instances of the subsoil blowing in due to the fluidity of the silt under the river bed, but it was possible to maintain the caissons in the correct positions and truly vertical.

The practice had been to keep the sand fill above high-water level, tending to hold up the caisson by friction, and it is stated that the tendency for the skin friction to hang up the caisson was a cause of the subsoil blowing in during the subsequent dredging. Reducing the level of the sand outside the caisson to within 10 ft. to 20 ft. of the river bed was suggested as a means of accelerating penetration, and this was adopted, but it was not considered desirable to increase the head of water in the dredging wells to build up positive pressure since the caisson had not been designed for such an increase of internal pressure, while higher lifts of concrete, which would have added weight for sinking, were not adopted due to the possibility of deterioration in the quality of the concrete.

The sand-island method was also used for the new road bridge over the Mississippi at New Orleans.\(^{10,12}\) The depth from mean water level to the bottom of the main pier foundation, which is in the river, is 180 ft. and only a little less for the other main pier. Because of the silty nature of the river bed and the consequent difficulties expected in preventing the caisson from tilting and the difficulty in correcting it, an open-dredge caisson was used in conjunction with
Fig. 155.—General Dimensions of Bridge at New Orleans for the Foundations of which the Sand-island Method shown in Fig. 154 was Used.
partial filling with sand of a sheet-pile enclosing cofferdam. A mattress 300 ft. by 500 ft. composed of woven willow sunk to the river bed and the caisson was sunk through it. The depth of sinking in this case exceeded the maximum of 115 ft. below water at which compressed-air work is permitted in the U.S.A. It is of interest to note the methods used for other bridges over the Mississippi. For the Baton Rouge bridge, open dredged caissons with sand-islands were also used. Open dredged floating caissons were used for the bridges at Natchez and Greenville. For the Memphis bridge compressed air was used for the river piers. For the bridge at Vicksburg pneumatic caissons were also used, but the Huey P. Long bridge, as described in the foregoing was the first instance of using open dredge when sunk through sand islands. In this instance steel cables were used to strengthen the mattress during sinking. Sixteen anchor cables each about 1200 ft. long were used to moor it to concrete blocks placed on the river bed. The mattress

Fig. 156.—Sinking the Steel Shell for the "Sand-island" Method.

was sunk by distributing stones on it. Next a steel sheet-pile U-shaped pen was placed around three sides of the site of the pier. Thirty-nine 36-in. diameter steel tube piles, each 150 ft. long, were driven 60 ft. into the river bed to support the pen. The tubes were filled with sand and gravel. The floating base for the caisson was fabricated 1900 miles away on the Ohio river and towed to the site. On arrival it was extended vertically an additional 27 ft. to give a total height of 47 ft. above the cutting edge. It was manoeuvred into the open downstream end of the pen and against the vertical beams on both sides and at the upper end. The downstream opening was then closed by a horizontal beam. Two guide beams were then driven vertically to complete the vertical slot within which the caisson could move only vertically. The procedure for sinking was then to place concrete in the space between the enclosing cofferdam and the dredge tubes followed by raising the walls of the enclosure and dredge tubes, followed by dredging with clamshell bucket until further raising of the caisson was needed and possibly without restricting access for concreting and dredging. It took
about two months for the cutting edge to reach 30 ft. below the river bed with 80 ft. of sinking still needed. The level of the water was kept the same inside and outside of the caisson. There are 6800 cu. yd. of concrete in the seal and the total quantity of concrete in the caisson is 35,000 cu. yd. A 10-ft. distributing block of concrete was constructed to cover the dredge-walls. A special type of varnish was used on the outside of the caisson to reduce friction, but as no measurements were possible of the friction without the varnish, the benefit could not be determined exactly.

**Penetration into Rock.**

When it is necessary to anchor deep foundations as well as support heavy loads, say because of hydrostatic uplift on the structure when the subsoil is saturated, a recent development of the cylinder method is to extend the excavation into rock beyond the bottom of the cylinder. This has been done by churn

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**Fig. 157.—Driving 30-in. Diameter Tubes 100 ft. to 116 ft. to Rock.**
drilling into the rock after the upper strata have been pumped out so as to give, on subsequent concreting of the socket in the rock, a mechanical bond that will resist uplift forces. An example of this method is the driving of 21 cylinders

(Fig. 157) each 29 in. internal diameter, for an extension of an electricity generating station. (10,13) The tubes were driven by a 90-ft. pile frame, using a single-acting steam hammer having a 7500-lb. ram and 39-in. stroke, with a special driving head to penetrate from 100 ft. to 116 ft. to rock through varying soft strata as
CONSTRUCTION OF CYLINDERS AND OPEN CAISSONS

1. AS LAUNCHED

2. BEING TOWED

3. SHORTLY BEFORE LANDING

4. EXCAVATION

5. PLAN OF CAISSON-WORKING DOCKS AND ANCHORAGE SYSTEM

6. COMPLETED PIER

Fig. 159.—Method of Sinking Caissons for the West Channel Piers of the San Francisco-Oakland Bay Bridge.
shown in Fig. 158 (a). The loads supported were as great as 1356 short tons on a single column, varying sizes of structural steel core being welded together to make up the necessary length and lowered into the tube to make a central steel core of one piece [Fig. 158 (b)]. The penetration through the clay and silt was about 20 blows per foot, but somewhat slower through the sand. Subsequent tests on the bond between the concrete in the socket and the rock face showed no slip at a bond stress of 386 lb. per square inch. The cylinder was kept full of water to prevent a blow-in, and the sedimentary soil was cleared from the inside of the pipe by means of a baler and sand pump. The latter consisted of a 15-ft. length of 12-in. pipe with a flap at the bottom, the jerking up of a piston in the pipe providing sufficient suction to draw the sand and clay into the pipe which was then hoisted and emptied at the surface. The churn driving was done with a 50-h.p. petrol engine, using a 4000-lb. cross shaft bit, 28 in. diameter, alternately drilling and baling out the broken rock. After drilling a short distance the cylinders were driven a further 6 in. to 12 in. into the rock, using a crawler crane and a smaller steam hammer, so as to seal off the inflow of ground water before concreting.

Deep Open Caissons.

It is evident that, when open caissons are sunk in deep water, buoyancy is required if they are to be towed to position; buoyancy is also necessary if they are lowered between temporary stagings while being built up in order to keep to practical limits the load supported by the stagings. This buoyancy may be obtained in the case of steel construction by the space for subsequent concrete between the inside and outside strakes, and with reinforced concrete sometimes in the same way, but more often the sides are solid without an annular space and buoyancy is obtained by a false bottom, for example as shown in Fig. 149.

The objections to the use of false bottoms, particularly in deep water, were overcome in the sinking of open caissons for the San Francisco–Oakland Bay bridge (10,14) by the use of a different and novel method by which the cells of the caissons were provided with domes and the buoyancy provided by entrapped compressed air, injected into the cells as required to maintain the desired submergence of the caissons during building up. The method is shown diagrammatically in Fig. 159 for the case of the caisson (W6) which provides the foundation for the main pier nearest to Yerba Buena Island and involved a depth to rock of about 216 ft. This caisson consisted of 28 cells, while that for the central anchor pier (W4) of the West Channel crossing (Fig. 161) was provided with 55 cells of the same size at 17-ft. centres, and the same method was followed for four of the main piers of this section to reach depths to rock varying from 107 ft. to 240 ft. through considerable thicknesses of mud.

The method greatly facilitates control of the caisson at the time it is brought into the correct position for sinking, since it can be promptly sunk into the channel bed by quickly releasing air from the cells before tidal currents may get it slightly out of position. Also during sinking (Fig. 160), control of the air pressure in groups of cells enables the caisson to be more easily kept vertical.

The following extract (10,15) describes the construction and sinking of the caisson (W4), the largest ever constructed, and which is typical of the other three.
Fig. 160.—Caisson being Sunk for a Pier of San Francisco-Oakland Bay Bridge.

Fig. 161.—Caisson for Central Anchor Pier, San Francisco-Oakland Bay Bridge.
The caisson consisted of a rectangular structure divided by longitudinal and transverse partition walls of reinforced concrete spaced on 17-ft. centres. These are carried by girders 12 ft. deep resting on a steel cutting-edge section 5 ft. 5 in. deep. The outer faces are covered with diagonal timber sheeting attached to suitable steel framing. In each of the 15-ft. square wells is inserted a steel tubular lining, 15 ft. in diameter, of 4-in. plating, and these are connected by adapter sections to the longitudinal and transverse girders above the cutting edge. The tubular linings are in suitable lengths, welded together and airtight. They thus form interior shuttering, between which and the outer timber sheeting concrete may be poured to form the structure of the caisson as it is continued upward to keep the top above water level during sinking operations. All or any of the tubular wells may be capped by a spherical steel airtight dome and each of these capped cylinders may be independently supplied with compressed air.

"The caisson is sunk by releasing air and removing a few of the domed covers at a time, dredging in the uncovered wells, replacing the domes, pumping up the air, and continuing this cycle until the caisson is sunk to the stiffer material below the mud. Air pressures are then gradually reduced, all remaining domes removed, and dredging continued in all the wells in the usual manner until the caisson reaches bedrock.

"The steel cutting-edge section was riveted and welded together on launching ways to a height of 17 ft. 6 in. and 197 ft. by 92 ft. in plan. This was launched and moored alongside the wharf. A structural steel frame of I-beam wales with angle verticals and braces was then erected above the outside and cross caisson walls. Vertical 10-in. timbers carry the 4-in. caulked diagonal sheeting forming the outer shell. Inside this shell the 15-ft. diameter cylinders were carried up from the adapter sections and capped with steel domes to a height of 77 ft. 6 in. above the cutting edge. Concrete was placed in outside and cross walls to stabilise the caisson during its journey to the pier site.

"During this period two construction docks supported on piles were erected at the pier site north and south of the location. Each dock carried two 20-ton derricks mounted on independent tripod cylinder foundations. Twenty-six anchors were placed in the bay bottom at approximately 350 ft. from the caisson sides. These anchors, of which 64 of 100 tons and 24 of 125 tons capacity were used, are of reinforced concrete and are unique in size and in the method of sinking by means of powerful water jet systems incorporated in their construction and supplied through hose pipes with water at 300 lb. per square inch pressure.

"The caisson was then towed to the site and placed in position between the two docks. At this stage the draught was 20 ft. and the freeboard 57 ft. Anchor tackles were attached and sinking begun by adding 5-ft. to 15-ft. layers of concrete in the spaces between the cylinders and the outer walls, the air pressure being gradually increased to maintain a minimum draught. When the concrete was sufficiently hardened the caisson was lowered, by releasing air, to a minimum freeboard of about 15 ft., and steel and timber walls and reinforcement added in units of 10 ft. to 20 ft. in height. Air was then released in five to seven of the 55 cylinders, the domes cut off, cylinder extensions welded on, and re-domed at the higher level. Air pressure was increased in these cylinders and the cycle repeated until all cylinders had been extended. With this new height of structure, concreting and sinking were again continued. These processes were repeated until the cutting edge was within a few feet of the bottom. At this stage the caisson had a total height of 117 ft. 6 in. a draught of 63-1 ft., and a weight of about 39,700 tons. The air pressure in each cylinder required to float the caisson was 22 lb. per square inch.

"The caisson was grounded at high slack water on December 22, 1933, by gradually releasing air until about 6,000 tons of buoyancy had been destroyed. The operation occupied about two hours, during which careful observation and adjustment of the lateral position was maintained. The final location was within less than 1 ft. of the designed position. The cutting edge was finally 78-6 ft. below datum with an average mud penetration of 7-2 ft.

"Subsequent sinking brought the caisson into stiffer materials, when air pressures were released and all domes removed [Fig. 161]. Further dredging in the open
wells resulted in the caisson reaching its final depth at 210.2 ft. below datum. In the later stages of sinking, water jets under 300 to 350 lb. pressure were employed for cutting through the material between the dredging wells and cleaning beneath the cutting edge. Suction pumps and final digging with toothless dredging buckets removed broken jetted material and the last loose fragments of rock. Daily diving inspections in 220 ft. depth of water were made to ensure a clean rock surface before depositing the seal concrete. Owing to the difficulty of holding the caisson at a definite elevation during final cleaning operations, the area covered by the central 25 cylinders was first cleaned to bedrock and these cylinders filled with concrete up to 34 ft. above the cutting edge. Special bottom dump buckets were used to deposit the 8,200 cu. yd. of concrete in this seal."

The foundations for the Mackinac suspension bridge, Michigan, (10, 16) are carried down to 190 ft. below water level and were constructed in 1954 to 1956.

![Fig. 162.](image)

The piers for the two main towers at each end of the 3800-ft. central span rise 572 ft. above water and are supported on circular caissons (Fig. 162), each being 116 ft. in diameter and containing 2530 tons of structural steel, 330 tons of which are in the cutting-edge. To sink them through about 90 ft. of soil to the rock required about 37,000 cu. yd. of excavation in each caisson. The dredging well was 86 ft. in diameter. The 15-ft. space between the inner and outer shells are divided into eight watertight compartments, which were flooded to sink the caisson and as the level of the water was approached by the top, extra height was provided by prefabricated panels. When the cutting-edge penetrated the soil, sinking was achieved by filling the compartments with concrete, and dredging proceeded. When the rock was reached, the inner part of the caisson was also filled with concrete. The south anchorage-pier (Fig. 163), is 92 ft. by 44 ft. in
plan and the caisson contained twenty-one dredging wells each of 9-ft. diameter. This caisson was sunk to rock at 130 ft. below the surface.

The caissons of the piers in Figs. 162 and 163 were each guided in sinking by four towers, which were upended after towing to the site. Vertical and raking spuds driven through 20-in. diameter tubes anchored the towers in position.

The sub-structure of this bridge is unusual because of the large quantity (about 440,000 cu. yd.) of concrete placed underwater by depositing the aggregate and subsequently injecting grout comprising cement, sand, pulverised-fuel ash, and a proprietary additive. Most of the materials were placed from barges at a rate of 2500 tons per hour, using conveyors to deliver the materials to various areas within the caisson. By this method of concreting, the quantity which had to be handled by mixers was reduced by nearly two-thirds, and enabled the caissons to be rapidly stabilised in case of storms. The grout was placed at about 200 cu. yd. per hour by a self-contained batching and pumping plant on a barge 150 ft. by 50 ft., the cement and ash being delivered to it in covered barges from which they were unloaded pneumatically. The sand was unloaded from barges by a crane with a clamshell grab. In the case of one pier more than 4100 cu. yd. of concrete were placed on each of five consecutive days.

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CHAPTER XI

PNEUMATIC CAISSONS

Where foundations are required below water level and the soil is fine sand or mud, so that it is not possible to use the open caisson method because flow of the soil under the cutting edges would affect the support of neighbouring structures, or where obstructions, such as boulders, occur which cannot be removed by mechanical excavation, then the pneumatic caisson method must be adopted. By the pneumatic method the working chamber is under compressed air balancing, or slightly exceeding, the hydrostatic pressure, and the inward flow of soil and water is thereby prevented.

When timber caissons are used in the pneumatic method, the joints are packed with oakum. With steel caissons all joints subjected to air pressure must be well caulked. Reinforced concrete is frequently not proof against loss of air pressure through the concrete, but with the precautions mentioned later this is overcome and reinforced concrete is nowadays generally considered to be not only the best material for pneumatic caissons but is also normally the most economical.

As with open caissons, the caisson is sunk under its own weight, usually by building up as it penetrates or by adding permanent concrete filling between the inner and outer strakes, or wales in the case of double-wall steel caissons.

The air pressure in the working chamber is adjusted so as to balance, or slightly exceed, the water pressure at the depth to which the cutting edge has penetrated, and the cutting edge must be sufficiently below the inside soil surface to prevent serious loss of air through the soil. To ensure air-tightness when the shafts are of concrete the inside surfaces may be painted with a bituminous or other suitable paint for sealing concrete, and for the same reason the construction joints in those parts of the work subject to air pressure are as few as possible and carefully made.

Air Locks.

According to the size of the caisson, one to three shafts with air locks provide access for men to the working chamber, for the removal of excavated material, and for placing the concrete seal. There are two principal types of air locks for caissons: one is the type in which access is through the top of the lock and a derrick is used to lift the bucket right out of the lock with the air-tight door carried on the bail of the bucket. This type is known as the pot-lid type, and the lid is clamped down when the bucket is in the lower part of the shaft. With the other type, access is by a side door and the bucket is hoisted by an outside winch. In each case, for small caissons it is possible to have a combination material and man lock, the one air lock serving the two shafts, but for larger caissons it is more usual to have separate men locks and material locks. Sometimes an additional lock is provided for concreting, but more often this is an attachment to the material lock and a separate lock is used for the men.
Fig. 164.—Sliding-door Type Material Air-lock. Fig. 165.—Skip (or Bucket) for use with Air-lock in Fig. 164.

Fig. 166.—Pot-lid Type Combined Material and Man Air-lock, and Skip (or Bucket).
A typical material air-lock of the sliding side-door type is shown in Figs. 164 and 165. A combination material and man air-lock of the pot-lid type is shown in Fig. 166. In the majority of cases the air locks are made to suit a 36-in. diameter shaft.

Sinking of Pneumatic Caissons.

When a pneumatic caisson is resting in the soil, and before excavation commences, it may of its own weight have penetrated sufficiently to fill the working chamber with soil. For this reason it is preferable that the caisson should be light so that the cutting edges do not penetrate very much, and in this way enable good access through the shafts for commencing excavation. Occasionally, with a soft soil and a heavy caisson, the soil will not only fill the working chamber but come up the shafts as well, and thus lead to serious delay before a reasonable working face can be obtained for excavation.

The excavation is usually carried out by hand and the material removed by skips through the material shaft. It is sometimes possible to make use of the air pressure in the working chamber to blow out mud and wet soil through a pipe with the opening kept just below the surface of the mud, the material being ejected in this way with considerable force when the working-chamber pressure is of any consequence, and the wear on bends in the pipe then becomes one of the practical difficulties with this method.

The caisson is kept level during sinking by excavating close to the cutting edges where the greater penetration is desired. Generally the excavation is kept clear of the cutting edge so as to leave this embedded in soil and avoid loss of air. When the caisson has sunk to the desired level, and to prevent it sinking farther due to its own weight gradually overcoming the skin friction and the support from the cutting edges during the time the concrete seal is being placed, it may be necessary to prop the caisson fairly substantially from the soil up to the roof. If propping is to be expected, either the caisson roof must be designed strong enough for this to be done safely or the props will need to be placed at pre-determined positions immediately under the ribs in the roof. Generally, however, the bearing pressure of the soil will be the determining factor, and this will necessitate a relatively large number of props and adequately spreading the load from them on to the soil.

To assist sinking, the air pressure is often reduced during times when the men are out of the working chamber, the amount of the reduction being found by experience during sinking, so as to give as closely as possible the amount that can be excavated during the next shift.

Placing the Concrete Seal.

The principal difficulty in placing the concrete seal is to ensure it completely filling the working chamber up to the underside of the roof. The method sometimes adopted is to place the concrete in the normal way as for surface work, using a fairly stiff mix and benching up around the cutting edges working towards the centre, and finally, when the space is too restricted for further placing by hand, grouting up so as to fill the cavity left between the top of the concrete and the soffit of the roof. The air pressure is maintained to force the grout and kept on until it has fully set. This method necessitates a very dry concrete
Fig. 167.—Stages in the Sinking of Caisson for River Pier of Lambeth Bridge, London.

[Details of the caissons are shown in Fig. 168.]
and, notwithstanding the advantages of a drier concrete in other constructional work, a better method is the following. Vent pipes are placed from the roof of the working chamber to the open air at the corners of the caisson and other points away from the shafts. A wet mix of concrete is used for the seal, and this is placed continuously until the concrete is found to be rising up the vent pipes, the displaced air having escaped first. As the quantity of concrete to be placed for the seal is frequently fairly considerable, and is usually in a thickness of up to 8 ft., the heat generated in setting is considerable and raises the temperature in the working chamber by 30 deg. F. or more. The rise of temperature using a wet concrete is, however, definitely lower than when using a drier concrete.
Examples of Pneumatic Methods.

A typical example of the use of the pneumatic method in sinking steel caissons for bridge piers is given by Lambeth Bridge, the successive stages in the sinking being shown in Fig. 167 and the details of the caissons in Fig. 168. The bridge was constructed in 1930–1932 and crosses the river Thames in five spans, totalling 776 ft. in overall clear length between end abutments. The caisson foundations of the river piers were sunk through varying proportions of Thames ballast and London Eocene blue clay. The following extract is from a paper by Mr. G. L. Groves (11.1)* relating to the sinking of the caissons.

"Each of the four piers is founded on a single steel caisson [Fig. 168], which is a rectangular steel structure measuring 108 ft. 1½ in. by 37 ft. 1¼ in. over the cutting-edge plates, and is 15 ft. in depth, the working chamber, which was provided with three shafts, being 8 ft. high. The sides of the caisson are of ¼-in. plate and the sides and roof of the working chamber of ½-in. plate. Twenty plate girders, measuring 5 ft. over angles and set 5 ft. 1¼ in. apart, support the working chamber roof, which is further stiffened by diaphragms between the girders. Triangular diaphragms are inserted between the skin-plate of the caisson and the sloping sides of the working chamber. The total weight of steelwork per caisson is about 292 tons.

"Erection of each caisson was carried out immediately over its final position in the river at a level above all but exceptionally high tides. For this erection whole timbers were placed projecting inwards from the staging surrounding the site of the pier, and on these as supports horizontal timbers were laid to receive the cutting-edge sections. For lowering purposes, four pairs of heavy brackets (removed after the caisson was resting in the river bed) were attached to the longer sides of the caisson at a distance of 18 ft. from the corners. Leading up from these brackets were flat steel hangers, which effected the lowering of the caisson by an arrangement of jacks and crossheads. Special bearing piles in groups of six were driven opposite the position of each pair of lowering-brackets for the support of the jacks. Some concrete was placed in the haunches of the caisson outside the working-chamber walls before the caisson was lowered, and this brought the total weight up to about 480 tons, or 120 tons per jack. The manipulation of the jacks in lowering so large and cumbersome a load a considerable distance was an operation demanding special precautions, and it was given careful consideration beforehand. In general terms, the procedure was to have the two jacks on one side of the caisson moving together, but never simultaneously with the other pair, and to drop the caisson on each side but not more than 4 in. at a time. Each pair of jacks had its own pipe connection to the pump with a control-valve near the latter, and a similar valve was placed in each branch-pipe leading to individual jacks. The jacks were provided with screw-collars, and these were always kept within about half a turn of the locked position when the rams were not in motion. The rate of lowering was purposely kept low to avoid any possible excuse for mishandling or lack of complete control at all stages of the operation; it averaged about 4 ft. per hour.

"The caisson was kept flooded from the time it entered the water, two 8-in. valves being provided for the purpose at each end, just above the concrete in the haunches, and the erection of the temporary caisson, which was the next item of work to be undertaken, was put in hand as a tide-work job. The temporary caisson was in two tiers, the lower one 20 ft. and the upper one 21 ft. 6 in. deep. Each tier was built up of panels of steel troughing, the troughs running vertically. At both ends of the panels of the lower tier and at the bottom of those in the upper tier the troughing was welded to a 9-in. by 4-in. angle, and between the two tiers was interposed a 16-in. by 6-in. steel joist with its web lying horizontally. The inner flange of this joist provided the middle of three walings required for the internal framing of the temporary caisson; the other two walings consisted of pairs of steel channels cleated to the troughing. The great advantage of troughing used for the

* References thus (11.1) refer to Bibliography on page 244.
temporary caisson in this way was, of course, its inherent strength as a beam, which permitted of forming a dam for a tidal height of 41 ft. 6 in. with only three frames of timber. This economy of internal framing, besides reducing labour and material to a minimum, greatly facilitated the masonry work of the pier by virtue of the comparatively unrestricted working space that it afforded.

"The following is a description of the successive stages of work involved in sinking and sealing the caissons and building the piers.

"It was necessary to concrete the whole of the permanent caisson around and above the working-chamber before compressed air could be put on, and this concreting required that the flood valves should be shut and the caisson pumped dry. To counteract the buoyancy of the caisson in its unwatered condition, and to keep it pinned down to the river bed, a considerable load had first to be added, and for this loading it was decided to employ the ballast for the concrete. A timber deck was therefore laid down within the temporary caisson to form the floor of a ballast bunker, which covered the whole available area except for openings left for the air shafts, concrete chutes, and access; timber-walled spaces were formed within the bunker for the accommodation of two ¹⁄₄-cub.-yd. concrete mixers. The amount of ballast provided as kentledge varied in each caisson with the levels to which the cutting edge sank in the river bed, and with the probable heights of tides during the period of concreting, the least and greatest quantities required being about 1,650 and 2,200 tons. As the ballast was loaded into the bunker the penetration of the cutting edge naturally increased, but the greater depth of the caisson added to its displacement and demanded yet more ballast to overcome the resulting increase in its buoyancy. In the first two caissons dealt with (those nearest the shores) the working chambers were practically full of solid material before compressed-air working could commence, and the consequent restriction of space rendered the early stages of excavation tedious and unduly costly. The contractors accordingly obtained the engineer's sanction to place two bearing girders transversely across the working chamber of the two midstream caissons, where river bed levels were lower and ballast loading had, in consequence, to be greater than in the caissons for piers Nos. 1 and 4. These girders [Fig. 168] were placed approximately midway between the shafts. They were made 2 ft. deep over the flange angles, with wide bottom flanges, temporarily stiffened with timber struts, and were attached to the roof of the working chamber by means of 7-in. by 3-in. steel channels in pairs. The bottom flanges of the girders were set 2 ft. above the cutting edge. These girders undoubtedly reduced the initial penetration of the caisson to some extent, and they also had the desirable effect of keeping the caisson very level during the ballast-loading and free-air concreting.

"Taking average figures for the four caissons, the depth of sinking was 27 ft., of which the top 3 ft. was through ballast and the rest through London blue clay, the latter fairly soft for the first few feet and then increasingly harder. The average total excavation per caisson was 4,000 cu. yd. and the man-hours of labour expended upon it just under 10,000, representing an output of 0.4 cu. yd. per man per hour. Eight sinkers to each of the three shafts proved to be the number best suited to the capacity of the arrangements for disposal of the excavated material. The skips, of 1 cu. yd. capacity, actually loaded about 0.53 cu. yd. of solid material (averaged over ballast and clay). The winding was effected by electric crane, of which the bond passed through a gland in the upper cover of the material lock, the height of wind being about 60 ft. All three shafts were used for material, and sinking was continued night and day in two shifts of 10 hours net working time, except that on Saturdays a 7½-hour day shift and on Sundays a 7¼-hour night shift only were worked. The average excavation for a full day's work of 480 man-hours was 192 cu. yd., representing a sinking of 1.3 ft.; the maximum recorded output for any one day was 246 cu. yd., equivalent to 1.7 ft. of sinking.

"The working chamber was not excavated to its full depth all over its area until founding-level was reached, the cutting edge being kept embedded about 1 ft. in a berm of clay some 2 or 3 ft. wide. With this condition obtaining, it was found that the caisson could be lowered a convenient amount by reducing the air pressure to 3 or 4 lb. per square inch below the hydrostatic pressure. In the ordinary course the pressure was kept about 2 lb. per square inch above the hydrostatic pressure
while the men were working, and what may be termed the 'sinking effort' of the caisson in normal circumstances was approximately as follows:

<table>
<thead>
<tr>
<th></th>
<th>Tons</th>
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<tbody>
<tr>
<td>Dead weight of permanent and</td>
<td>3,100</td>
</tr>
<tr>
<td>temporary caissons, concrete,</td>
<td></td>
</tr>
<tr>
<td>shafts, locks, and timber</td>
<td></td>
</tr>
<tr>
<td>Average displacement of above.</td>
<td>1,300</td>
</tr>
<tr>
<td>(This varied to some extent with</td>
<td></td>
</tr>
<tr>
<td>the tidal height and with the</td>
<td></td>
</tr>
<tr>
<td>depth of the caisson in the</td>
<td></td>
</tr>
<tr>
<td>river-bed)</td>
<td></td>
</tr>
<tr>
<td>Balance</td>
<td>1,800</td>
</tr>
<tr>
<td>Displacement of working chamber</td>
<td>720</td>
</tr>
<tr>
<td>Balance</td>
<td></td>
</tr>
<tr>
<td>Deduction for upward force of</td>
<td>1,080</td>
</tr>
<tr>
<td>2 lb. per square inch of air</td>
<td></td>
</tr>
<tr>
<td>pressure above hydrostatic</td>
<td>510</td>
</tr>
<tr>
<td></td>
<td>570</td>
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'Sinking effort' under working conditions.

With the cutting edge kept embedded as explained above, this downward force of between 500 and 600 tons was not sufficient to sink the caisson, and the latter was 'blown down' during the men's meal times as frequently as the progress of excavation demanded, the conditions then being,

<table>
<thead>
<tr>
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<th>Tons</th>
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<tbody>
<tr>
<td>'Sinking effort' under working</td>
<td>570</td>
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<tr>
<td>conditions, as above</td>
<td></td>
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<tr>
<td>Add downward load resulting</td>
<td>1,520</td>
</tr>
<tr>
<td>from reduction of air pressure</td>
<td></td>
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<tr>
<td>by about 6 lb. per square inch</td>
<td></td>
</tr>
<tr>
<td>'Sinking effort' while 'blowing</td>
<td>2,090</td>
</tr>
<tr>
<td>down'</td>
<td></td>
</tr>
</tbody>
</table>

Under the action of this force of approximately 2000 tons, it was possible to lower the caisson at a steady, almost imperceptible, rate by as much as was required to provide a good volume of excavation for the next shift's work without unduly restricting the headroom in the working chamber.'

An example of shallow pneumatic caissons constructed on a staging and sunk to a shallow penetration to form the base of a quay wall is the reconstruction of Plantation Quay at Glasgow. The soil penetrated was boulder clay, mostly quite impervious but in places having pockets of sand and gravel. The caissons were each 70 ft. by 25 ft. in plan and consisted of a working chamber only so that the total height from cutting edge to roof was 10 ft. 6 in., giving a height inside of 7 ft. 6 in. The caissons were built on during and after sinking within a temporary caisson in the manner indicated in Fig. 169 and the caissons were lowered between guide piles placed against the longer sides of the caissons. The following extracts from a paper by Mr. T. J. S. Mallagh (11,2) give some details of the placing and sinking:

"After assembly at the building berth the launching of the caisson was accomplished by lowering it with four hydraulic jacks, which were mounted on steel girders supported by timber staging. The caisson was first raised 18 in. off the staging, and 50 tons of concrete were then placed in the bottom of the shoe to act as ballast during stage 3 [Fig. 169]. The staging beams were then removed, and the caisson lowered in 18-in. steps by the hydraulic jacks.

"All four jacks were operated simultaneously to ensure that the caisson was raised or lowered evenly, thus preventing any uneven distribution of the loading. The time required for lowering through a distance of 20 ft. was about three hours, the lowering being timed to take place on a flowing tide so that as the caisson was lowered the tide rose to meet it.

"The temporary caisson consisted of a number of strakes each from 16 to 20 ft. long and 5 ft. 2½ in. deep. The strakes were formed of ½-in. steel plates with 5-in. by 3-in. by ½-in. angles riveted along the four edges, and were bolted together with the angles on the outside to form the temporary caisson. The total height from the
cutting edge to the top of the strakes was 37 ft. 1/2 in., of which 26 ft. 6 1/2 in. represented the temporary caisson; at the back the permanent caisson was carried up to a height of 10 ft. 7 in. above the roof girders. The three bottom tiers of strakes were attached to the permanent caisson by means of 1 1/8-in. bolts each 16 ft. long, which fitted into nuts left permanently embedded in the concrete. The function of these bolts was to minimize the work of the diver when the strakes had to be removed.

"The temporary caisson was heavily strutted against external pressure by 12-in. timbers and walings, the vertical spacing of the struts being 5 ft. 3 in. and the horizontal spacing 8 ft. It was never necessary to strut the temporary caisson for its entire height, because before shoring was required at the top of the caisson it could be dispensed with at the bottom [Fig. 169, stages 4, 5, and 6]."

"In addition to being strutted, the temporary caisson was tied together by three lattice girders placed 10 ft. from the top of the strakes, and while in place these girders served to bind the strakes together and prevent any tendency to flexure. When the concrete reached the level of these girders they had to be removed, and in consequence the temporary caissons became less rigid. The shuttering of the concrete had of necessity to depend for its support on shoring off the temporary caisson, and as a result the freshly-deposited concrete transmitted an outward thrust to the strakes; in other words, the temporary caisson at certain definite periods was subjected to an external head of water and an internal head of wet concrete. At low water the internal head was the greater, and the lattice tying-girders having been removed, the temporary caisson tended to bulge outwards from its true position, thus allowing the shuttering to move out from its position, and also relieving the pressure on the short struts between the hard concrete and the strakes. (This is shown in stage 7.) These struts were only held in position by pressure, and in consequence when this occurred they dropped out. To overcome this difficulty ties were fitted at the top of the temporary caisson. This did not completely overcome the difficulty, for although the strakes were held rigidly at the top and the bottom they were still free to move at their centres, and no matter what was done to prevent it there was always some slight breathing of the strakes between high and low water. The effects of this movement were partially overcome, in the first place by fixing the short shores between the concrete and the caisson in such a way that when the pressure came off them they did not fall out, and secondly by placing concrete at the front of the caisson at high water only. Owing to the fact that the permanent caisson was carried up at the back this trouble only occurred at the front and to a limited extent at the sides.

"On the first occasion when flooding occurred considerable damage was sustained, the main struts and the shuttering for the concrete being displaced and damaged, and all freshly-placed concrete lost. After this experience it was decided to take additional precautions to minimize the damage if the caisson were again flooded, and accordingly the strakes were tied against internal pressure by wire ropes and unions, the struts being bolted in position so that they could not be displaced by flotation, and the height of the strakes being increased 12 in. by timber falsework. Further, in order that the liability to flooding might not be increased, the sinking of the caisson was postponed, but excavation was continued.

"On the second occasion that the caisson was flooded the damage was not extensive, but the repeated floodings had delayed the concreting whilst excavation had been progressing rapidly; the centre of the floor of the working chamber had been lowered well below the level of the cutting edge, and it was considered unsafe to proceed further until sinking could be resumed. Before the temporary caisson could be dispensed with, and sinking resumed, 600 tons of concrete had to be placed and a 4-ft. lift of new shuttering fabricated. This would, under normal circumstances, have taken eight or nine days, and so to minimize this delay continuous concreting was adopted.

"Unfortunately, shortly after the continuous concreting had been started, very boisterous weather set in, causing abnormally high tides and flooding the caisson on two more occasions. However, in spite of these delays the concreting was completed.
Fig. 169.—Successive Stages in Sinking a Pneumatic Caisson from a Staging.
in under five days, thus allowing the strikes to be dispensed with and normal working conditions resumed.

"The maximum amount of concrete placed in one day was 96 cu. yd., but the average rate for placing it, including time spent on shuttering, but not including other delays, amounted to 36 cu. yd. per day. This was considered quite satisfactory, as the shuttering was of a complicated nature, and some of the concrete had to be handled twice by cranes.

"When the caisson had been concreted to a height of 14 ft. 9 in. over the cutting edge it only floated at or very near high water, and it was calculated that it required an additional weight of 150 tons to retain it permanently on the bottom. As it was not practicable to supply this weight in the form of kentledge, the only alternative was to place 150 tons of concrete between two successive high waters.

"In the case of the first two caissons everything was carried out according to programme, and no unforeseen difficulties arose, the procedure being as follows. On a morning high water, the caisson was floated into its correct position and held there by mooring chains. If on the ebbing tide the caisson grounded in a satisfactory position, the concreting was immediately started and not stopped until enough had been placed to prevent the caisson refloating. As the tide ebbed and the caisson rested more heavily on the ground it canted away from the old quay without, however, the cutting edge moving from its correct position. In the first caisson this cant reached a value of I in 31 within a few hours of the caisson grounding. This cant decreased each day, and by the time excavation was started the caisson had become level. In the second caisson the cant reached a value of I in 34 at the time of grounding, but gradually increased and when excavation was started it had reached a value of I in 20. Neither of these caissons showed any tendency to slip out of position.

"The third caisson was ready for setting permanently on the ground on September 7, 1932. It contained approximately 940 tons of concrete, and the falsework was in position to take the next lift. It was decided to set the caisson in position on the morning tide of September 8, and it was hoped to have 1,100 tons of concrete in the caisson by that evening. At 5:30 a.m. the caisson floated, and at 7 a.m. it was moored in its correct position. Concreting was then started, and at 7:30 a.m. the caisson grounded in a satisfactory position. At 8:15 a.m. the caisson had canted away from the shore but the shoe remained in the correct position, the cant reaching a maximum value of less than I in 165. The position of the caisson was kept under observation till 8:30 a.m., and as no tendency to move was observed it was assumed that the caisson would remain permanently in its correct position; at 2 p.m., however, it was noticed that the caisson was rather far from the shore. On checking its position, it was found that it had moved 9 in. bodily into the river, so the placing of the concrete was stopped immediately. The caisson then contained 1,040 tons of concrete, and it was hoped that it would be possible to refloat it on the afternoon tide, but unfortunately this proved to be impossible.

"Two alternatives were possible: the first was to wait until the Spring tides due in a week's time in the hope that an extra high tide would float the caisson, although it was very doubtful that a high enough tide would occur. The other alternative was to cause the caisson to float by means of some external agency. After consideration, it was decided to try to float the caisson by means of air pressure. Two man-locks were temporarily fixed on the short access shafts and their open tops were blocked off by steel plates, everything being ready for applying the air pressure by September 12. It was calculated that an air pressure of only 11 lb. per square inch would be necessary to cause the caisson to float and, as 30 lb. of air could probably be obtained, no difficulty was anticipated. What had to be guarded against, however, was the tendency of the caisson to turn turtle once it floated. Fender blocks were made to go between the inner guide-piles and the caisson, and these blocks were of such a size that the caisson would lie 3 in. inside its correct position when moored against them.

"High water on September 12 was at 1 p.m. and at 12 noon the air pressure was turned on, and was increased to 14 lb. per square inch without the caisson moving. At 1 p.m. the assistance of a ten-ton and a five-ton crane was obtained
in order to try and induce the caisson to float, and at 1.30 p.m. it very reluctantly began to leave the bottom and the top was hauled over against the fender blocks, the shoe, however, remaining 9 in. outside its correct position. At 2.40 p.m. the caisson had moved so that the shoe was 3 in. inside its correct position and the pressure was then released, with the result that the caisson cantled over towards the river. The cant reached a value of 1 in 150 in a few minutes, and then the shoe slid out into its former position. By this time the tide had ebbed a considerable amount, and it was decided to wait till the next day before trying to float the caisson again.

"The next attempt to set the caisson was made on May 13, and meanwhile 8 in. were taken off the inner fender blocks and additional fenders were made for placing between the caisson and the outer guide-piles at river bed level. Air pressure was put on at 9 a.m. and the caisson showed signs of moving at 11.30 a.m., but did not properly float till high water at 1 p.m.; when it had been hauled in tight against the guide-piles, the outside fenders were placed in position at river bed level by the diver, and at 2.30 p.m. the air pressure was released, with the result that the caisson first cantled a little and then slid out to within 1 in. of its correct position. On the next day the cant of the caisson had reached a value of less than 1 in 310, and this subsequently increased to 1 in 120 before excavation was started.

"In the case of the fourth caisson the same trouble occurred, but to a lesser extent. As the result of experience gained on the third caisson several precautions were taken. Two sets of fender blocks were made, one set for the outer guide-piles and one for the inner, these blocks being of such a size that when in position the caisson would lie 1 in. inside its correct position. This caisson was ready for setting on November 2, and it floated at 3 a.m. on the following day. The fenders were in position at 5 a.m. and shortly afterwards the caisson grounded in a satisfactory position, and the placing of the 150 tons of concrete necessary to hold it permanently on the ground was started at 5.30 a.m. The caisson remained quite level and showed no signs of moving till 6 a.m., but shortly after this time it was noticed that the caisson was out of position. Concreting was immediately stopped, fortunately only about 10 tons having been placed. On taking observations it was found that the caisson was 1 ft. out of position. At 3 p.m. the caisson refloated, and in the meantime the fenders had been removed and 8 in. taken off the inner fenders and 8 in. added to the outside. The caisson was placed in position and held there by the new fenders, the outside fenders being placed at river bed level by the diver. At 5 p.m. the caisson grounded and lay 9 in. inside its correct position with a cant 1 in 60 shorewards, and about half an hour later the caisson slid out to within 1/16 in. of its true position. The concreting was then resumed and continued through the night, and by the next morning 130 tons of concrete had been placed and the caisson had moved to within 1/16 in. of its correct position.

"When the caissons had been sunk to their final levels the average error from the correct line over the four caissons was under 2 in. The worst caisson was the second, being 3 in. too far out, and the best was the third, which was 1/16 in. too far in.

"Excavation under air pressure was started in the first caisson when only 5 ft. were required to complete the concrete substructure; that is, when there were 2,500 tons of concrete on the caisson. The total downward load of the caisson was then 2,840 tons and the upward displacement due to an air pressure of 20 lb. per square inch was 2,240 tons, thus giving an excess load of 600 tons for sinking. The theoretical pressure required to overcome the hydrostatic head at high water was only 15 lb. per square inch, but 20 lb. was actually the maximum pressure reached at this stage of the sinking. The supply of air was maintained at a pressure of 15 lb. per square inch for 12 hours before the working chamber was cleared of water, and it was then found that the slurry of the river bed had risen about 2 ft. up the access shafts. Progress was very slow at first as only one man could work in a shaft at a time, and it was a week before men could enter the working chamber, and another five days elapsed before the excavation of the boulder clay was started.

"To avoid the recurrence of this delay in the subsequent caissons, the excavation was started when about 16 ft. of concrete was required to complete the substructure, the weight of concrete on the caisson then being 1,700 tons, while for
safety the air pressure was regulated so that there was always an excess downward load of at least 200 tons. This arrangement permitted much more rapid work at the beginning of the excavation, but there was still a certain amount of delay due to the presence of slurry in the working chamber. The best progress was finally made by fairly heavy blasting, using pneumatic spades to break up the displaced clay. Excavation, and hence sinking, never reached the estimated rate, this being mainly due to the exceedingly tenacious nature of the boulder clay, and also to delays caused by blows and other causes. However, the experience gained in sinking the first caisson led to improved progress in those subsequently sunk."

Fig. 170 gives particulars of the rates of sinking. "The maximum rate of excavation was 20 skips per hour, but this rate could only be maintained for very short periods, a good average rate being reckoned at 10 skips per hour. Assuming

![Graph showing rates of sinking]

Fig. 170.—Rate of Sinking Caisson shown in Fig. 169.

that each skip held \( \frac{1}{2} \) cu. yd., 50 cu. yd. could be excavated in a 10-hour shift with both shafts working. The average amount actually excavated per shift amounted to about 12 cu. yd.

"The total weight of the caisson with the substructure built to full height was 3,000 tons, and to balance this weight an air pressure of 30 lb. per square inch would have been required. However, the usual minimum pressure of 15 lb. per square inch, added to the skin friction and the resistance of a bank of clay at the shoulder of the cutting edge, easily prevented uncontrolled sinking. When it was desired that the caisson should sink the air pressure was reduced to about 5 lb. per square inch and the caisson would sink under perfect control.

"In general, the method of controlling sinking was as follows. Levels were taken on the top of the substructure every morning and from these the level and inclination of the shoe were deduced. During the day the bank around the cutting edge was trimmed so that it tended to correct any deviation of the caisson from the plumb. Each evening, at the change of shift, the pressure was reduced to allow the caisson to sink, and in sinking it generally returned towards the plumb."
Fig. 171.—Swing Span of Bridge on Pneumatic Cylinder Caissons shown in Fig. 172.
Compressed air was used for the six cylinder caissons forming the foundation of the main pier of the swing span of the Kincardine-on-Forth road bridge shown in Figs. 171 (a) to (c) and Fig. 172; the arrangement of the access through the air locks is shown in Fig. 173. The total load of the moving span is 1600 tons, and the total load on the six cylinders including the piers themselves is 4200 tons. It was originally intended to construct the 42-ft. diameter cylindrical pier from the rock level within a cofferdam. However, the cofferdam proved subject to blows and, after being seriously damaged in an unsuccessful attempt at dewatering, the method of construction was modified to sinking the six cylinders shown.
and then constructing the pivot pier on top as a hollow in-situ reinforced concrete cylinder as originally designed. The following extract relating to the cylinder

**DIAGRAM OF MAN AND MATERIALS AIR-LOCK.**

**DIAGRAM OF 14.6" DIA. STEEL CYLINDER CONVERTED FOR USE WITH COMPRESSED AIR WITH WATER USED AS KENTLEDGE.**

**COMPRESSED AIR CYLINDER AS ADOPTED AT PIER NO. 11 WITH CONCRETE USED FOR KENTLEDGE.**

![Diagram of man and materials air-lock and diagrams of 14.6" dia. steel cylinder and compressed air cylinder as adopted at pier no. 11 with concrete used for kentledge.]

Fig. 173.—Typical Examples of Air Locks for Pneumatic Caissons.

foundations of the swing span is from a paper by Mr. Guthrie Brown (11,3) describing the design and construction of the bridge.

"The steel shuttering for the 14-ft. 6-in. diameter cylinders was already available at the site, being surplus from the 100-ft. span piers then completed, and the proposal was to sink the cylinders for the centre pier under compressed air into solid rock in a similar manner to the 100-ft. span piers. In order to avoid the necessity of bringing forward kentledge for the cylinders it was decided to use the permanent concrete to give the required load against uplift. The steel cylinders were supported from three pairs of jacks, and perforated link-plates with large diameter pins were provided to carry the load while the jacks were released and reset. The whole load was carried on temporary staging, and as concreting proceeded within the steel cylinders they were gradually lowered into the water to obtain a reduction by water
displacement of the total load on the jacks. By this means the whole of the steel cylinder with its encasing concrete and the internal air-lock shaft in position was lowered on to the river bed, when excavation under air was put in hand until a sufficiently stable support had been obtained to permit the jacks supporting the load to be dispensed with. Excellent foundations on sandstone of particular toughness were obtained for all six cylinders at depths of about — 38 O.D., these being keyed about 3 ft. or 4 ft. into the rock. The maximum air pressure experienced was 21 lb. per square inch. The working chamber and central shaft were then concreted up to the level of the underside of the slab at — 12 O.D. A few lengths of steel sheet piling were driven to seal the gap between the cylinders, and the whole of the inner space was filled with gravel to this level to simplify the construction of the slab. The base of the slab is 3 ft. below water, but the cofferdam was sufficiently tight to withstand a few feet difference in head, and the slab was thus constructed in the dry without difficulty. The construction of the pier occupied eight months from the date when the first of the six cylinder foundations was approved for concreting."

**Pneumatic Method with Ground-water Lowering.**

The limitations of using the pneumatic method when the depth exceeds 100 to 115 ft. below the free-water surface may sometimes be overcome where it is possible to lower the ground-water level by pumping. The method is obviously restricted to suitable surface and subsoil conditions by which the volume of water to be handled is reasonable but it can, for example, be used in the bed of a river if a cofferdam protects the site of the caisson and an impermeable stratum separates the river bed from the artesian water. Lowering
the subsoil water level could be effected in that case by an artesian well, or wells, otherwise pumping from the caissons is the alternative.

Instances of the successful use of this method have been cut-off walls for dams where the depth has been too great for the driving of sheet piles. In the case of the Quabbin reservoir the caissons were sunk through 120 to 130 ft. of water-bearing gravel, sand, and boulders; as the ground-water level was lowered some 90 ft. it was possible to do most of the sinking in free air and for the balance the air pressure required did not exceed 25 lb. For the Merriman dam (114) of the Delaware water supply project, of which the caisson cut-off wall is seen in Fig. 174, the same method was used for sinking caissons 38 ft. and
45 ft. by 10 ft. to 15 ft. in plan through an overburden of water-bearing and impermeable soil, sand, gravel, clay, glacial fill, and boulders to bedrock at a maximum of 160 ft. below the bottom of the cut-off trench.

The first two caissons, 38 ft. by 15 ft. in plan and 180 ft. high, were sunk as exploratory caissons, and in the details of these shown in Fig. 175 will be seen the sleeves for four well-points at about every 20 ft. vertically, so inclined that pumping of isolated water-bearing soil layers could be carried out after the caisson was sunk and so assist in the sinking of subsequent caissons. For sinking the exploratory caissons pumping was done from the working chamber, and for all caissons the general subsoil water level was lowered by an artesian well.

The ground-water level was lowered in this way as much as 120 ft., enabling the majority of the work to be done in free air and maximum air pressure not to exceed 32 lb.

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CHAPTER XII

FLOATING BOX CAISSONS

Box Caissons as Breakwaters.

If box caissons are designed to form sections of a breakwater or a quay wall and should need to be towed some distance, then in addition to the normal requirements the considerations which affect the design of ships may become of importance, in particular the hogging and sagging moments, the torsional stresses in a rough sea, and the towing resistance.

These and other similar considerations arose in the case of the "Phoenix"

reinforced concrete box caissons (Fig. 176) made in England and towed to the Normandy coast for forming the breakwaters enclosing the greater part of the two principal ports for the initial stages of the invasion of the Continent in 1944. Descriptions of the harbours have been given elsewhere, but it may be of interest here to make a few remarks that could be used for guidance in future cases where box caissons are being considered for use under somewhat similar conditions. The writer is indebted to Brigadier Sir Bruce White, M.Inst.C.E.,

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then Director of Ports and Inland Water Transport, War Office, for permission to give the following brief particulars, and more complete data are now available. (75)

It should be noted that, although many fears were expressed at the time this scheme was formed of the useful life that could be expected of a breakwater founded in this way on the surface of the sea-bed, the results more than vindicated the acceptance of the inevitable risks and difficulties in carrying out an operation of such magnitude in an area of military operations. The units were of six sizes to suit the varying depths of water, the largest type being that shown in Fig. 177.

Methods of Launching.—The units were constructed in a variety of ways according to the sites and the contractor's choice. The smaller units were generally partly built in the dry on slipways; but some of the larger units were completely built in wet docks which had been dammed and then dewatered, the dock being refilled when the units were practically complete.

* References thus (13-1) refer to Bibliography on page 250.
METHOD OF TOWING.—Towing was generally by means of a towing bridle of 4-in. wire cable with spring shock absorbers (Fig. 178), the Senhouse slips being provided for easy casting off. The towing resistance in still water was about 10 tons for the largest type (A1) units for a towing speed of 5 knots.

HULL STRESSES WHEN FLOATING.—The considerations are the same as those of a ship except that the righting moment reduces during towing when the sea passes over the gangway. Since there is a normal freeboard of 5 ft. to 8 ft. for the gangway, the unit is only slightly less responsive in rising to head waves than if it has a ship shape with flared bows. During sinking, this reduced width above the gangway causes a temporary list of some 7 deg. towards the side on which the gangway first becomes submerged.

TORSION.—Owing to its rectangular shape in plan the caisson was subject to greater torsion in a cross sea than if it had been ship shape. The torsional shear resulting from a heavy cross sea was adequately resisted by the concrete U-shape cross section of the caisson, but diagonal steel tie-bars were provided across the open tops of the cells to distribute any build up of unbalanced torsional stresses consequent on founding on an unfavourable bottom.

SINKING PROCEDURE.—Generally, sinking was effected by opening the valves (X) in Fig. 177 by means of handwheels placed at deck level, with extended shafts. These valves were 12 in. and 7 in. in diameter and a total of twenty was provided to the largest size units.

FRICTION AND PENETRATION INTO SEA-BED.—Aerial photographs of the harbours taken at various dates show that some of the units shifted on the sea-bed. This was probably due to the resistance to penetration of the sea-bed, and would not be likely to occur under average conditions where there was some penetration. Sliding did not give more than slight trouble on these sites.

HULL STRESSES AFTER FOUNDING.—The hull stresses when the caisson is founded may be very different from those when it is floating, and a ship of normal proportions is generally subject to considerable hogging moment (which may be accentuated by scour) if it is beached in shallow water; in fact this is more
than anything else the cause of the early breaking up of wrecks in shallow water. Caisson units are similarly affected if provided with swim ends, and the reinforcement in the top of the side walls needs to be calculated accordingly.

**Prestressed Concrete Cap on Box Caisson.**

The Spit Bridge at Sydney (12.2) is shown in the frontispiece. The piers (Fig. 179) comprise a reinforced concrete headstock, on four 16-ft. diameter cylinders, supporting the bascule-span and the operating machinery. The cylinders were sunk as pneumatic caissons to rock at some 70 ft. below the bed of the river. The tops of the cylinders are about 4 ft. 6 in. below mean sea level; the headstock imposes a weight of 2000 tons of concrete. The headstock is 9 ft. deep, 45 ft. wide, and 84 ft. long and is solid at the upstream and downstream ends which are connected by four massive reinforced concrete beams. The tidal range at the site is 6 ft. so that the underside of the headstock is always under water. The possibility of constructing a cofferdam was considered, but owing to the size and depth required, the alternative of floating out a prestressed concrete shell for the headstock was adopted. The cylinders were capped with concrete slabs, and a 4-in. hollow rubber ring was placed around the outer periphery of each cylinder to form a seal so that the water pressure under the tank could be released on placing and to facilitate subsequent grouting of the space between the tank and the top of the cylinder. Steel plates 2 in. thick, drilled and tapped to take 2-in. dowel bars, were set and anchored into and 1 in. below the top of the cylinder wall and the space was packed with lime-cement mortar. The design provided for cutting holes through the 64-in. floor of the tank and screwing the dowel bars into these plates after the tank was in its final position and the gap beneath had been grouted. Redesign of the longitudinal girders in post-stressed concrete allowed the two outer girders to be reduced in width from 6 ft. to 1 ft. 10 in. and the two inside ones from 3 ft. to 1 ft. To obtain the required buoyancy, it was necessary to place a temporary timber floor under them between the tanks. The outer main girders were prestressed with nineteen 14-in. Macalloy bars and the inner girders with eight 14-in. bars. The bars were sheathed in flexible tubing before being cast in place. The girders were designed as prestressed concrete for positive bending moments and as reinforced concrete for negative bending moments. As the weight of the concrete member is greater at the ends considerable negative bending moments occurred in the beams after flotation.

The temporary timber floor of 5-in. hardwood planks spanning across the longitudinal beams was sheeted on the underside with mild steel plate for watertightness. The calculated weight of the precast tank was 450 tons and it was estimated to float with a draught of about 5 ft. There was a depth of 7 ft. of water at high tide over the top of the cylinders; this gave ample clearance for moving into place. For the floor the mixture of the concrete is \( \text{I}: \text{I-28}: 3 \) with a water-cement ratio of 0.44; the strength of cylinders was 5900 lb. per square inch at twenty-eight days. For the prestressed concrete girders the mixture was \( \text{I}: \text{I-33}: 2-66 \) and the strength from 5620 to 6370 lb. per square inch. The steel bars were initially stressed to 95,000 lb. per square inch and were grouted in after stressing, and after cleaning the ducts with compressed air.

Because construction of a slipway would require some excavation and much
under-water work, the dry-dock method was adopted, partly also because steel sheet-piling was available. A two-level dry dock, each 90 ft. by 50 ft., was constructed on the shore; the top level provided for the casting platform to allow work to proceed in the dry except with an abnormal tide. The lower
dock was excavated to enable floating off at a reasonable high tide, after removal
of the outer wall of the dock. The completed tank was floated off on a high
tide and moored for several weeks under the approach span of the new bridge
while awaiting the preparation of the seating on the cylinders and placing of
the rubber rings. The rings were designed to compress \( \frac{1}{4} \) in. on placing the tank,
to allow for irregularities in the tank floor. While moored the shuttering was
put in place for casting the central deck between the solid ends. The tank was
placed at high tide. It was manoeuvred into its approximate position using two
launches, and finally located by winches set up on the tank. Four 2\( \frac{1}{4} \)-in. mild
steel dowels were then dropped through the pipes set in each corner of the tank
and the tank manoeuvred slightly until the dowels fitted into recesses in the tops
of the cylinders. As the tide fell, the tank slid slowly down the dowels, minor
adjustments being made by the winches. The final setting was obtained using
two theodolites set at right-angles to line up on sighting marks on the tank walls.
A diver was used to check the final seating of the tank. The rubber rings were
filled with a concentrated brine solution to help maintain them on the top of
the cylinders. As the tank settled on the rings, valves opened to allow escape
of the solution and compression of the rings.

After seating, the two central compartments at each end were filled with
fresh water and the temporary timber floor in the central section was breached.
This additional weight at the ends and loss of uplift on the central section was
sufficient to prevent the tank refloating at high tide. Breaching of the timber
floor was necessary to prevent overstressing the central longitudinal beams if
the tank had been subjected to a greater head of water than that necessary for
flotation since the consequent negative bending moment would add to the negative
bending moment due to the prestressing. The central deck concrete was first
to be placed and enabled the water to be emptied from the end compartments.
The whole work of concreting and cutting of false floors had to follow a strict
routine so as not to exceed the permissible stresses in the various parts of the
structure under the various loadings of wet concrete and uplift at high tide.

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CHAPTER XIII

SUB-AQUEOUS TUNNELS

METHODS of constructing tunnels under waterways are described briefly in this chapter since one method is similar in procedure to floating into position and sinking caissons, and another method involves a cofferdam and constructing the tunnel in the dry. The three principal methods (Fig. 180) of constructing under-

**SHIELD METHOD**

Holland tunnel in New York (double)
Lincoln tunnel in New York (double)
Scheldt tunnel in Antwerp

**UNDERWATER METHOD**

Oakland tunnel (Cal.)
Meuse tunnel in Rotterdam

**METHOD OF OPEN EXCAVATION**

Railway tunnel at Velsen
Motor tunnel at Velsen

Fig. 180.

water tunnels are (a) by underground excavation, usually with compressed air and shield; (b) by sinking precast sections of the tunnel into a prepared channel in the waterway; and (c) by providing a cofferdam and constructing the tunnel in the dry and in open air. In the past, the method (a) has been mostly used. Method (b) was used for a tunnel at Rotterdam, for Hampton Roads tunnel and
Precasting and Floating into Position.

Clearly the bed of the waterway is a preferable foundation either naturally if there is good soil for the support of the tunnel without risk of scour or settlement, or if means can be taken to remedy any deficiency in either respect. To meet these requirements, using method (b), and also to avoid hazards from above, it is usual to dredge the bed of the waterway and to replace later the dredged material over and around the tunnel to provide a cover giving protection against ships' anchors and possible future scour or dredging. One way of providing a satisfactory bearing where the surface exposed by dredging is silt or soft clay is to dredge deeper and replace by sandy soils as mentioned on page 210.

A recent example of a sub-aqueous tunnel constructed by floating and sinking tunnel sections on a prepared dredged bed is the Hampton Roads Crossing, Virginia, U.S.A. The main water crossing is about 3 1/2 miles long and short lengths of hydraulically-placed embankments were formed at both shore ends. The tunnel under the main channel is 7479 ft. long. The two islands forming the tunnel portals are artificial, and each contains an open approach and ventilation building. The roadway in the tunnel is 23 ft. wide and provides one lane of traffic in each direction. The tunnel itself is composed of twenty-three units each 300 ft. long sunk into place and backfilled with a minimum cover of 5 ft. of sand. The maximum depth from mean sea-level to the top of the tunnel is 80 ft. and to the bottom of the dredged trench about 120 ft. The construction of the tunnel units commenced with the making of a steel-plate tube 33 ft. in diameter, surrounded by a polygonal steel-plate trough connected by steel diaphragms to the tube. The trough contained a concrete envelope placed outside the shell by means of which the buoyancy and sinking was controlled. Steel bulkheads were provided at each end, and when enough concrete had been added as ballast the unit was towed from the fabricating yard at Chester, Pa., to a fitting-out pier adjacent to the site. Additional concrete placed in the outer casing reduced the buoyancy to be just sufficient to keep a unit floating for towing to the actual position. The prepared trench was covered by 2 ft. of pea gravel carefully screeded to the correct line and gradient. Each unit was positioned during sinking by sighting on steel towers erected near the end of each unit and of sufficient height to project above the water when the unit was sunk in its final position. For the sinking operation, two large barges were rigidly connected by steel framing from which falls were used, while additional concrete was placed under water by tremie in the outer envelope. When in position on the bottom, large steel pins were placed by divers, to connect the unit to the adjacent unit which had been sunk previously. Steel semi-circular closure plates were inserted in grooves at each end of each unit and driven to the bottom of the trench to act as shuttering for the exterior concrete junction between the units.

Borings showed silt to a depth of nearly 100 ft. below water level in the vicinity of the north portal island, where the depth of the water did not exceed 20 ft. Construction of the island would cause compression of the silt over a period, resulting in a primary settlement of up to 5 ft., and a secondary settle-
ment of about 2 ft.; such settlements were not permissible. Seven schemes were studied, taking into account possible damage by off-course ships and extreme floods in hurricane, before the construction of an island was selected, and the sand-island method, involving removal and replacement of the silt, was adopted. The construction of one of the islands is shown on Fig. 181; a typical section through the tunnel is shown by Fig. 182, two of the precast tunnel units being fitted out are shown in Fig. 183; and the interior of the tunnel during the construction of the roadway is shown in Fig. 184.

The Deas Island tunnel (13.1) under the Fraser River in British Columbia is another example of a sub-aqueous road tunnel constructed by sinking precast elements in a channel dredged in a river bed. The land on both sides of the river at Deas Island (Fig. 185) is extremely flat, low-lying and little developed. The lowest point of the tunnel lies about 78 ft. below these areas, whereas for a bridge it would have been necessary to raise the roadway about 156 ft. above them. The tunnel solution thus gives an appreciable saving in fuel. It was possible to limit the length of the tunnel to 2160 ft., and this permitted a considerable saving in operational costs for ventilation and illumination. There is a ventilation building on either side of the river at the transition between the approaches and the tunnel. The maximum gradient of the tunnel is 4.5 per cent. The navigable depth above the tunnel is 42 ft. at low tide, the tunnel being sunk in a trench dredged across the river-bed. After each element had been sunk on to concrete blocks providing temporary support, sand was jetted into the space beneath it. Sand filling, protected by gravel and rock, was placed on both sides of the tunnel, and the top of the tunnel was protected with rock (Fig. 186). In order to protect the river-bed on both sides of the tunnel against scour, the
protection provided by the sand and rock was supplemented by prefabricated concrete mattresses reinforced with stainless steel and articulated longitudinally and transversely. The mattresses were laid as a continuous blanket across the river and were held in position by rock filling. The banks were given a similar protection both above and below the line of the tunnel.

Fig. 182.

Fig. 183.
Fig. 184.

Fig. 185. Location of Deas Island Bridge.

Fig. 186.

Fig. 187.
Figs. 187 and 188 show cross-sections of the tunnel. The rectangular cross-section was considered preferable to a circular section because the diameter of the latter is determined by the width of the roadway. For a given navigational clearance, the level of the roadway for a circular cross-section is lower, which entails deeper, longer, and more expensive approaches. The small height of 23 ft. 6 in. enables the tunnel to some degree to follow possible differential settlements without cracking. Another important advantage was that the depth of the dry dock in which the tunnel elements were cast could be limited correspondingly, which was of great economical importance.

The outside of the tunnel elements was provided with a waterproofing membrane. Under the bottom the membrane consisted of \( \frac{1}{16} \)-in. welded steel sheets. On the top and sides a bituminous membrane was used, made up of four layers of glass fibre reinforced in hot asphalt.

The tunnel elements were constructed in a dry dock dredged for the purpose north of the river and near the line of the tunnel. The sub-aqueous tunnel was built up of six elements, each of which was 344 ft. long, 78 ft. wide and 23 ft. 6 in. high, and weighed about 18,700 tons. The bottom of the dry dock was 633 ft. by 384 ft. so that it might contain the six elements simultaneously. The dry dock was dredged using a suction dredge after the upper layers of silt had been removed by means of drag-lines and used for a dyke surrounding the dry dock. On completion of the dredging the channel connecting the dry dock and the river was plugged, and the dry dock dewatered. The ground water within the entire area of the dry dock was lowered to at least 7 ft. below the bottom of the dry dock by means of a two-stage system of well-points.

The dry dock was flooded when all the elements were ready for floating. At intervals of two to four weeks they were towed to an outfitting jetty where steel towers were fixed to facilitate aligning the units while being sunk. Control of movement up and down stream was provided by special anchors of \( 3100 \) lb. weight, 70 ft. long, and 6 ft. wide, with steel-plate flukes and jetted about 30 ft. into the mud of the river bed. Sinking each unit involved about 6 hours to warp it approximately into position when the central ballast compartments were
flooded, and subsequent operations of sinking and horizontal movement. Final vertical alignment was obtained by 300-ton jacks and movable steel columns fixed at each corner of each unit. The joints between the units were sealed by inflated rubber gaskets; the permanent connection was affected by welded plates and concrete filling.

**Use of Cofferdams.**

Method (c) was used for the road and railway tunnels under the North Sea canal at Velsen, between Ymuiden and Amsterdam. A cross-section of the tunnel is shown in Fig. 180. The work was done in two stages with cofferdams with the centre of the waterway enclosed in both sections as shown by Fig. 189. Since the canal had been previously widened elsewhere, the work within the southern cofferdam could be done without reducing the then existing usable width of the

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**Fig. 189.**

**Fig. 190.**

**Fig. 191.—Excavation for Velsen Tunnel.**
canal at that point. After completion of that stage, and with the cofferdam removed except for a central island, the waterway was formed to the south, and construction of the northern cofferdam commenced. A typical section through the south excavation is shown in Fig. 190. A view of the excavation for the second stage is shown in Fig. 191. The soil exposed in the excavation was fine sand, with a stratum of clay at a depth of 45 ft. with a stratum of peat immediately below. The stratum of clay separated the brackish sub-soil water above from artesian fresh water below, and since it was important not to allow the two types of water to mix, as it would spoil the local supplies of drinking water, the method of dredging and of constructing the tunnel was chosen with this in mind. The pumping of the upper brackish water was done by centrifugal pumps to a depth of 26 ft. below the level of the water in the canal.

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Dravo Corporation.—Fig. 103.
Economic Foundations Ltd.—Figs. 12 and 13.
Frankipile Ltd.—Fig. 16.
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Messrs. Lazarus White and Edmund Prentis. Figs. 77, 98, 99, 121 and 122.

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