SHEET PILING COFFERDAMS AND CAISSONS

BY

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CONCRETE SERIES

LONDON

CONCRETE PUBLICATIONS LIMITED

14 DARTMOUTH STREET, LONDON, S.W.1

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"CONCRETE SERIES" BOOKS
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First Published (3000 copies) . . 1945
Reprinted 1949

Made and Printed in Great Britain by Butler & Tanner Ltd., Frome and London

PREFACE

SOONER or later all civil engineers are likely to be concerned with deep foundations, but whereas there are recent works on the theory and practice of bearing piles, in the case of sheet piling, cofferdams, and caissons the writer has felt the lack of a concise modern treatment that could be read with interest and also serve as a reference book. At the same time an effort has been made to include the results of recent theory and practice on matters which are of importance to both the designing engineer and the resident engineer.

It will no doubt be generally agreed that in numerous cases in the past unnecessarily expensive methods have been used and economical or more satisfactory alternatives have been overlooked; therefore the examples have been selected with a view to emphasising the reasons influencing the choice of method or type of construction.

Perhaps for the first time types of cofferdams and caissons are shown diagrammatically so as to give the reader a more comprehensive picture, and it is believed that this new approach to the subject will be helpful. The writer hopes that the book will serve a purpose, even if only by drawing attention to the uses and suitability of new, and some of the lesser known, methods of design and construction.

The writer's thanks are due to those who have kindly lent illustrations or given their views on one or other of the controversial questions discussed in this book, and to Mr. C. E. Riches for his valuable help in the development of the calculations for sheet-piled walls and in the checking of proofs.

D. L.

June 1945.

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PART I

SHEET PILING

General.

SHEET piling consists of a line 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.* 1. 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 driven in soft soils may often be driven with square ends without damage, but if the piles are at all large or long, or the soil may necessitate hard driving, a sheet-metal protection ($\frac{1}{16}$ in. or $\frac{1}{8}$ in.) must be provided to form a pointed edge. Should there be need to penetrate compacted gravel or a stratum of shale, shoes of cast iron as shown in *Fig.* 2 (*a*) may be necessary. A shoe with a pointed edge with one face only sloping is usual in trench excavation. For permanent construction, only a few varieties of timber are suitable if used in waters where timber is subject to attack by marine borers. Generally speaking, marine borers

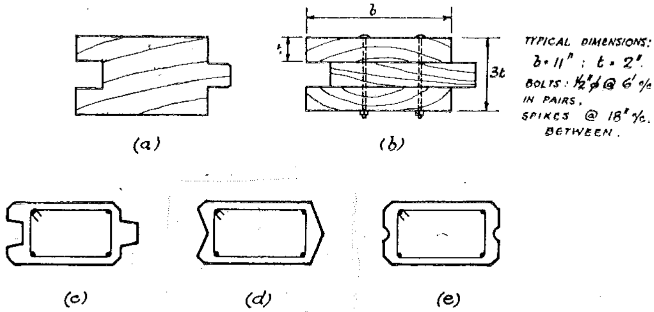


FIG. 1.—TYPICAL SECTIONS OF TIMBER AND CONCRETE SHEET PILES.

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

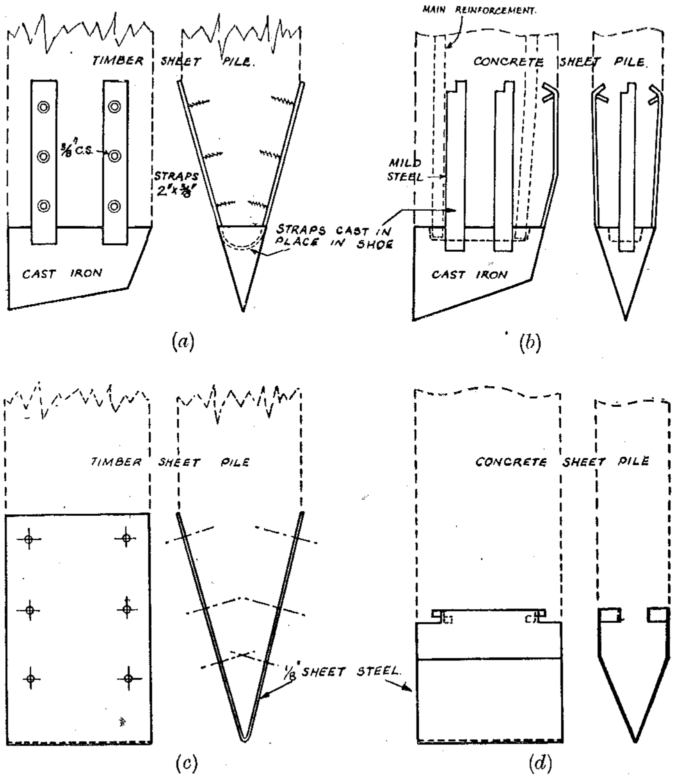


FIG. 2.—TYPES OF SHOES FOR SHEET PILES.

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.

REINFORCED CONCRETE SHEET PILES

Typical sections of reinforced concrete sheet piles are shown in *Fig.* 1. 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, much to be preferred as it enables better alignment of the sheeting to be obtained.

Table I gives the properties of a selection of sheet piles, and typical details of the reinforcement are shown in *Fig.* 3. The detail shown in *Fig.* 3 (*a*) was that used for Dunston power station,[1] while that shown in *Fig.* 3 (*b*) is a detail prepared by the writer to suit type 10B of *Table* I.

TABLE I.

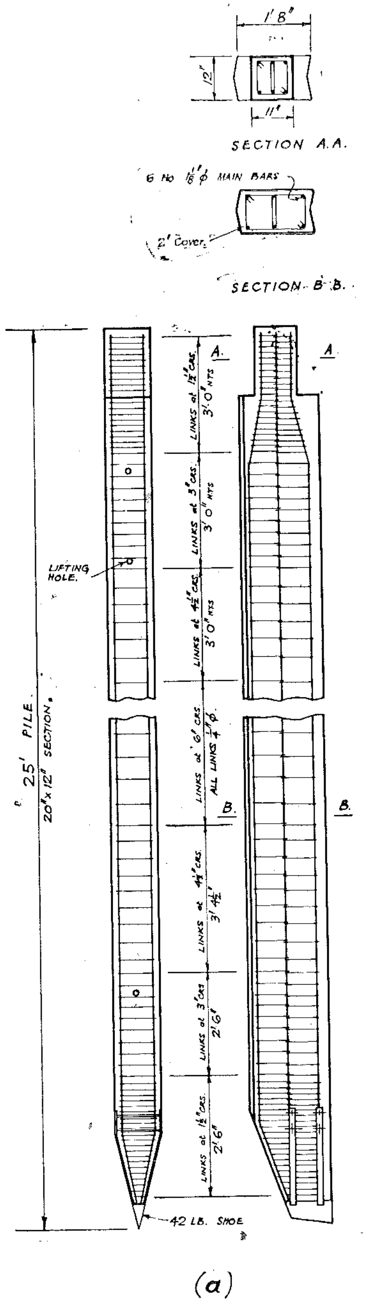
PROPERTIES OF REINFORCED CONCRETE SHEET PILES.

Reference	5A	5B	6A	6B	7A	7B	8A	8B
Main bars	2—½″	2—½″	2—⅝″	2—¾″	2—¾″	2—¾″	2—¾″	3—¾″
Width (in.)	12	9	14	16	16	14	15	18
Thickness (in.)	5	5	6	6	7	7	8	8
Resisting moment per foot width in pounds-feet	1,900	2,530	3,200	3,950	4,800	5,480	6,040	7,520

Reference	9A	9B	10A	10B	11A	11B	12A	12B
Main bars	3—¾″	3—⅞″	3—⅞″	3—1″	3—1⅛″	3—1⅛″	3—1⅛″	3—1⅛″
Width (in.)	18	20	20	20	22	20	20	16
Thickness (in.)	9	9	10	10	11	11	12	12
Resisting moment per foot width in pounds-feet	8,700	10,500	11,900	15,500	19,800	21,800	24,300	30,300

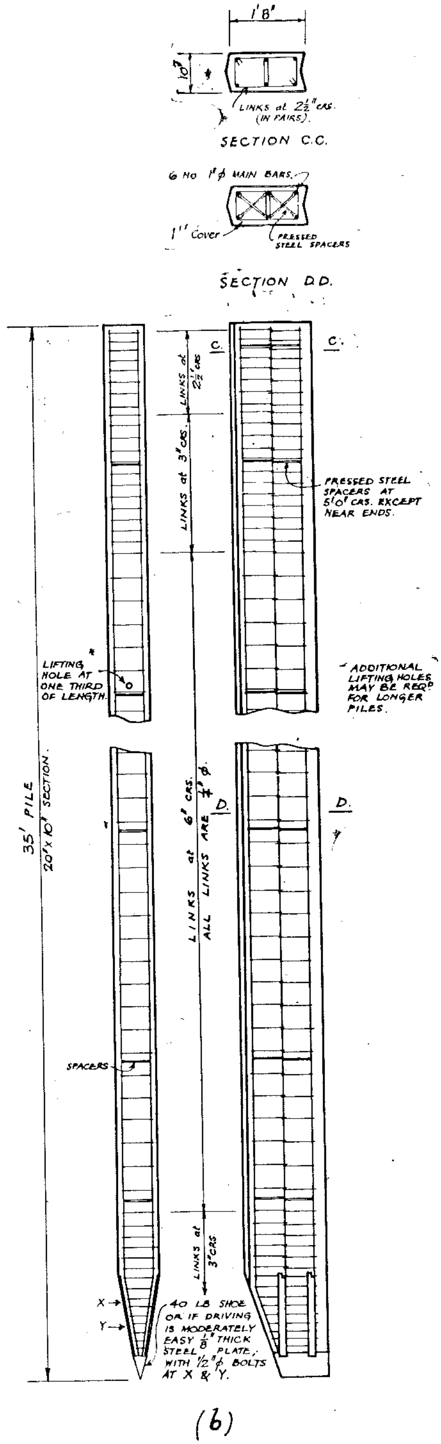
The steel listed occurs on each face. Stresses: Concrete 750 lb. per square inch; Steel 18,000 lb. per square inch.

It is both usual and the best practice to reduce the heads of reinforced concrete sheet piles to take a driving helmet, but this is not essential if 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.



(When driving, the sloping edge of the shoe is on the far side from the piles already driven. The pile first driven, however, is symmetrical, and is normally slightly sloped on both edges.)

Fig. 3.—Typical Details of Reinforced Concrete Sheet Piles.



With concrete sheet piles to be driven in soft soil a shoe of sheet metal (16 gauge up to $\frac{1}{8}$ in.) is often sufficient. 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.* 2 (*b*) is used.

Materials and Proportions.

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 : $1\frac{1}{2}$: 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 mix near the head, but this increases the time of maturing and it also delays the stripping of the pile shuttering and the handling of the piles. Where piles are not required to be driven as soon as possible, a mix of 1 : $1\frac{1}{2}$: 3 is used with ordinary Portland cement, and both the shrinkage stresses on the reinforcement and brittleness of the concrete are reduced by this method.

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 ultimate strength of the pile. The stresses during driving are dealt with in "Reinforced Concrete Piling," by F. E. Wentworth-Sheilds and W. S. Gray.

The requirements of the Code of Practice for the Use of Reinforced Concrete in Buildings [2] can be applied to the design of concrete sheet piles except that experience shows it is not necessary to provide as much as the minimum percentage of lateral binding which is rightly suggested to be necessary for buildings. For concrete bearing piles and normal driving conditions, the volume of lateral binding to that of 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 in the middle part of the length of the pile, and enormous numbers of piles have been driven into very hard soil with a percentage of binding only slightly exceeding these values.

For concrete sheet piles it is, however, sometimes desirable to increase slightly the percentages over that for bearing piles, since with the rectangular cross-section of sheet piles the binding is generally less effective. Nor is it necessary with slender concrete piles to limit the maximum load to the percentages of the permissible axial load for short columns given by the Code of Practice and shown as curve C in *Fig.* 4. Provided that bending stresses are properly taken into account the curve C can be used for low values of l/k, but otherwise curve R may be used because it allows more for eccentricity where it is most needed, viz. below $l/k = 60$. The stresses for the higher values of l/k may be considerably increased as shown by curve R; this is of importance with laterally unsupported bearing piles and thin concrete sheet-piled walls, and can be justified by theory as well as by practice.

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 tying together the main reinforcement it is important that the main bars are not permitted to come too close to the surface, both because of risks of spalling of the 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.

The amount of concrete cover over the main bars is a compromise between

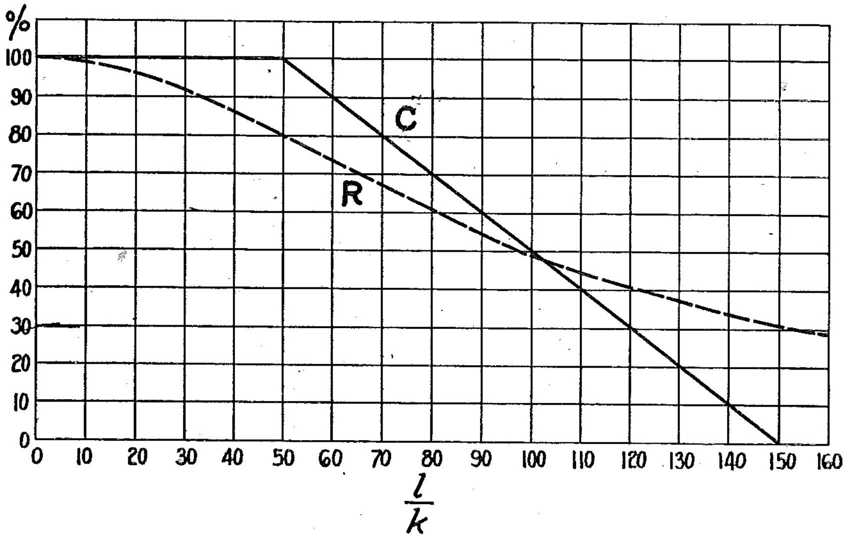


FIG. 4.—PERMISSIBLE MAXIMUM AXIAL COMPRESSION STRESS IN SLENDER REINFORCED CONCRETE COLUMNS AND PILES.

cost and effectiveness. For concrete sheet piles extra cover is expensive and for piles in fresh-water $\frac{3}{4}$ in. may be sufficient, with a concrete mixture of $1 : 1\frac{1}{2} : 3$ and good workmanship and driving conditions. For sea-water a cover of $1\frac{1}{2}$ in. over the main bars is usual for the same conditions.

Special Types.

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 West-Rotinoff piles shown in *Fig.* 5 and used for this purpose for the Assiut Barrage, Egypt.[3]

Stretcher bars were cast 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 plumb. The steel mandrels extended for 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 $2\frac{1}{4}$ in. The plant used for sinking the piles consisted of a No. 21B Ruston-Bucyrus crawler navvy, with a 40-ft. jib for handling and pitching the piles and for operating the 2-ton driving monkey. 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.

As the top $19\frac{1}{2}$ in. of the piles were housed in the new concrete floor, the

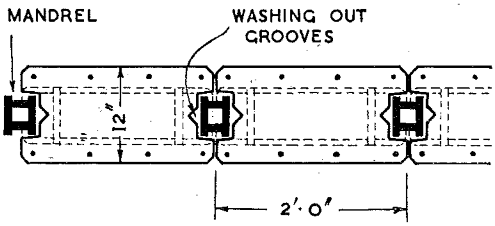


FIG. 5.—SPECIAL TYPE OF REINFORCED CONCRETE SHEET PILES USED FOR A CUT-OFF WALL IN THE RECONSTRUCTION OF THE ASSIUT BARRAGE, EGYPT.

excavation for the new work had to be more or less completed before the piling was carried out.

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

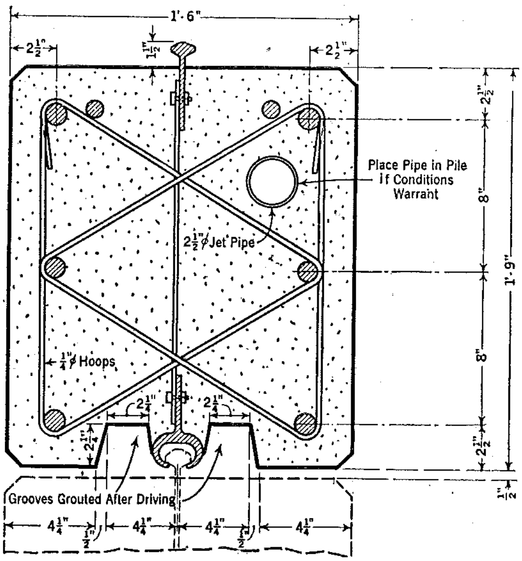


FIG. 6.—A SPECIAL TYPE OF REINFORCED CONCRETE SHEET PILE USED FOR A CUT-OFF WALL NEAR THE RIVER MISSISSIPPI.

occasional tap with the monkey if required. The sinking of the last 6 ft. 6 in. was carried out by means of the monkey only, the jetting pipes being held 6 ft. or so above the bottom of the pile.

Water-jet pipes were lowered into the grooves in front of and behind 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 connection was then fitted to the top of the mandrel leading to the grouting 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.

Another example of a special type of reinforced concrete sheet pile is shown in *Fig.* 6. This unusually heavy type of pile was used for the Cahokia power station, St. Louis,(4) 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 steel pile interlocks being subsequently grouted to prevent seepage.

STEEL SHEET PILES

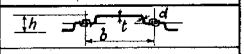
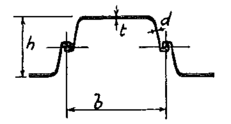
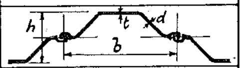
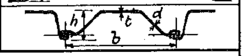
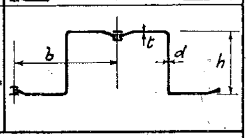
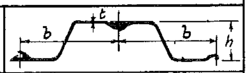
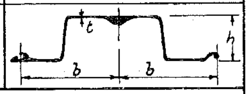
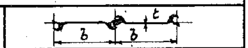
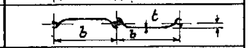
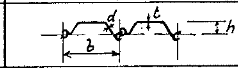
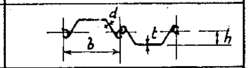
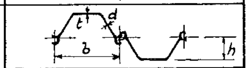
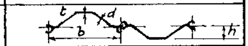
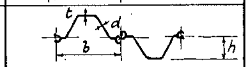
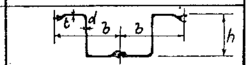
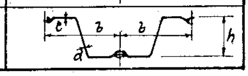
In *Table* II is shown a selection of steel sheet pile sections with their properties. Most of these sections are frequently rolled but, as sections are sometimes listed by makers although rolls have not been made, enquiries should be made while new work is in the design stage. The Carnegie sections shown are typical for America, and in many cases the Bethlehem sections are almost identical.

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, and recently a writer (5) was much criticised for suggesting that the modulus of the assembled wall is about 75 per cent. of the combined section, so that in effect the friction in the interlock transmits most of the longitudinal shear. The sections referred to were, however, of the American type, in which there is a good deal more clearance and flexibility in the interlocks than in the European sections.

With the latter types 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 few feet 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.

TABLE II.

TYPES AND PROPERTIES OF TYPICAL STEEL SHEET PILES.

SECTION	b ins.	h ins.	t ins.	d ins.	WEIGHT lb/sq.ft. WALL	MODULUS in³/ft. WALL	PROFILE.
LARSSEN							
0 G B	10 13/16	2 15/16	0·20	0·20	11·47	2·2	
1 G B	15 3/4	5 1/8	0·32	0·23	18·50	7·8	
1 U	15 3/4	5 1/8	0·375	0·375	21·70	9·1	
2	15 3/4	7 7/8	0·41	0·31	24·98	15·8	
3	15 3/4	9 3/4	0·55	0·35	31·74	25·3	
4B	16 9/16	13 1/2	0·63	0·43	40·90	42·5	
5	16 9/16	13 1/2	0·87	0·47	48·74	55·1	
10A	17 3/4	7 7/8	0·50	0·50	27·30	11·7	
2/10A	15 3/4/17 3/4	4 13/16	0·41/0·50	0·81/0·50	26·30	6·9	
DORMAN LONG							
K S I *	16·93	6·30	0·37	0·37	21·71	12·31	
K II	15·75	7·87	0·32	0·32	24·78	19·75	
K III	15·75	9·45	0·43	0·35	32·56	31·29	
K IVa *	15·75	11·02	0·55	0·36	38·09	43·34	
K IV *	15·75	11·02	0·63	0·39	41·17	46·69	
K Va *	14·17	12·60	0·67	0·41	48·54	60·46	
K V *	14·17	12·60	0·79	0·47	54·27	66·77	
FRODINGHAM							
I A	15 3/4	5 3/4	0·27	—	18·25	10·09	
II	15 3/4	7 9/32	0·32	—	24·17	18·53	
III	15 3/4	9	0·42	—	31·50	26·42	
U.S.S. CARNEGIE—ILLINOIS							
M 107	15	—	3/8	—	31·0	3·0	
M 108	15	—	1/2	—	35·0	3·1	
M 112	16	1 11/32	3/8	—	23·0	2·4	
M 113	16	1 11/32	1/2	—	28·0	2·5	
M 117	15	3 5/16	3/8	3/8	31·0	7·1	
M 106	14	3 1/8	3/8	3/8	31·0	8·9	
M 110	16	6	31/64	3/8	32·0	15·3	
M 115	19 5/8	3 7/16	3/8	3/8	22·0	5·4	
M 116	16	5	3/8	3/8	27·0	10·7	
MZ 38	18	12	1/2	3/8	38·0	46·8	
MZ 32	21	11 1/2	1/2	3/8	32·0	38·3	

* ROLLS NOT NECESSARILY AVAILABLE.

Questions of the efficiency of the interlock naturally do not arise with symmetrical sections, and only for fractional stresses in cases where the clutches are on the outer faces.

The modulus is given in *Table* II, in each case from the maker's tables, but in the writer's opinion the American method of discounting completely friction

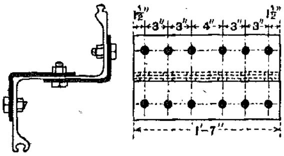


FIG. 7(*a*).

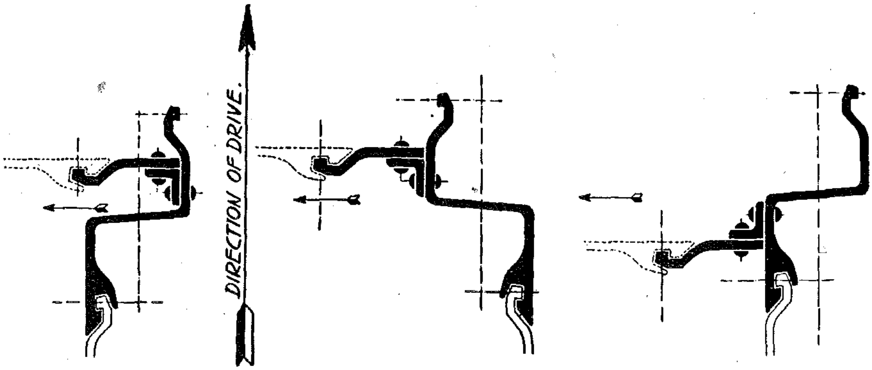


FIG. 7(*b*).

FIG. 7.—METHODS OF FORMING JOINTS IN STEEL SHEET PILES.

in the interlock is too conservative, just as the opposite applies slightly to European sections with the modulus 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 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.* 7(*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 the Dorman Long type of sheet piling the interlock, although rolled separately, is normally welded at the foot to the sheet pile on one side, and in common with several other systems of sheet piling the piles are driven in pairs.

When the headroom is restricted, the upper part of the clutch of the leading edge of the piling, as 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 other pair of piles and, in pitching, is threaded over the exposed tongue of the leading edge of the last driven pair of piles, so that when the driving is completed the halves of the clutch abut. It will be noted that when this is done the piles are driven with a clutch leading; normally a clutch is driven over the tongue of an interlock of a previously-driven pile.

There is normally no real loss of strength in jointing the clutch where the clutch acts merely as a locking device between adjacent sheet piles, but there is a serious loss of strength if this method is used with types of piling where the clutch supplies a large part of the transverse strength.

Corrosion of Steel Sheeting.

The corrosion of steel sheeting 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,[6] the pollution of river waters and tidal waters, particularly by sewage and industrial waste, leads to widely varying degrees of attack. An approximate basis of the rate of corrosion is $\frac{1}{300}$ in. per year in sea-water and $\frac{1}{500}$ 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 if the steel is left exposed to rust 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 will then usually be sufficient to prevent from becoming active the pin-head size scale particles which remain just below the surface. Sand-blasting is now used exclusively by the Bridge Department of the State of California for the maintenance of bridges, and where, as with steel sheet piling, subsequent maintenance is facilitated by a soundly-adhering first coat, this method deserves the increasing attention it is receiving. For maximum durability, after thorough cleaning the steel should be primed with a 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[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. 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 with exposed surfaces does not apply if water and air have free access.

In recent years much has been said of the advantages of including 0·25 to 0·33 per cent. of copper in the steel to reduce corrosion, but tests by the L.M.S. Railway [8] lead the writer to the conclusion that though the copper content is normally worth the small extra cost, it cannot be considered an alternative to proper painting.[74]

If the appropriate corrosion rate previously mentioned is divided into half the web thickness of any sheeting section it is intended to use, its probable life will be seen to be ample for many uses without any particular surface protection.

Under-water Cutting of Steel.

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.

TABLE III.

OXYGEN PRESSURES FOR CUTTING STEEL SHEET PILING UNDER WATER.

Depth	Heating oxygen pressure (lb.)	Acetylene pressure (lb.)	Cutting oxygen pressures (lb. per square inch) Material thickness	
			½ in.	1 in.
Surface	100	½	30	50
5 ft.	105	5	35	55
10 ,,	110	10	40	60
15 ,,	115	15	45	65
20 ,,	120	20	50	70
25 ,,	125	25	55	75
30 ,,	130	30	60	80

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. The oxygen pressures required for heating and cutting are given in *Table* III, and typical arrangement of connections and regulators are shown in *Fig.* 8, which applies to a fuel-gas supply of either hydrogen or acetylene for cases where the cutting is not more than 30 ft. below the water surface. The gases are conveyed to the cutter by flexible rubber pressure tubing having connections secured by worm-drive clips, the tubes being taped together at intervals for convenient handling by the diver, who should preferably be used to flame cutting in the open air before attempting under-water cutting. The procedure is to light the cutter before the diver goes down. To do this, the valves on the outlets of each regulator are first opened, while the cutter valves are closed. The valves on each cylinder are then opened very slowly by lightly tapping the keys by hand, avoiding a sudden rush of high-pressure gas. The high-pressure gauges then show the pressure in each cylinder. The pressure-adjusting screw is slowly turned until the low-pressure gauge indicates the pressure required from each regulator as indicated in the table. The heating oxygen valve on the cutter is then opened, next the fuel-gas valve, and the flame is lit. In the case of hydrogen the flame should be about 4 in. long, while with acetylene there should be a fairly well-defined cone

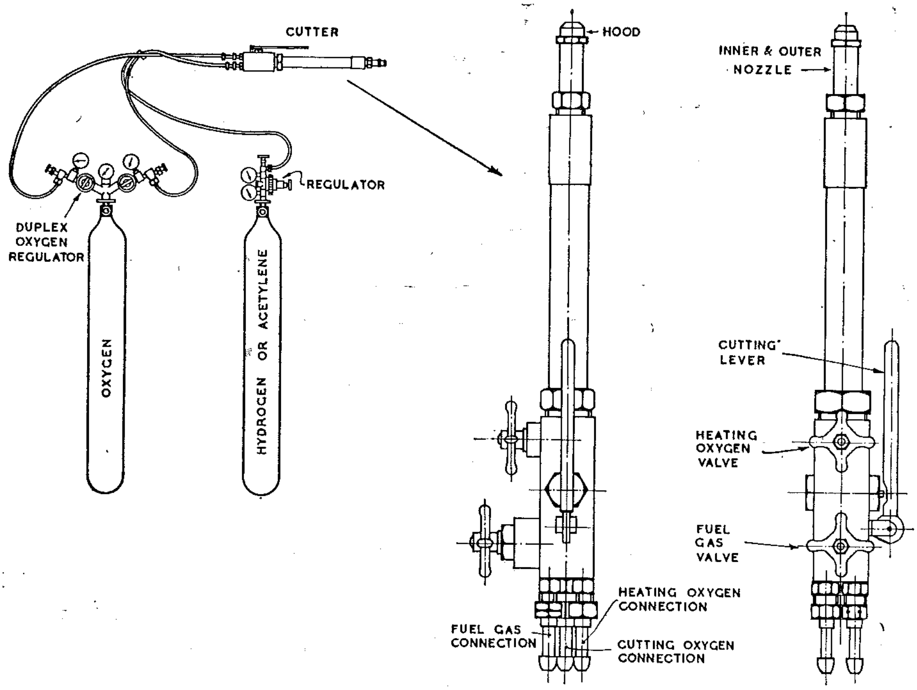


FIG. 8.—APPARATUS FOR CUTTING STEEL UNDER WATER UP TO 30 FT. BELOW THE SURFACE.

only about $\frac{1}{2}$ in. long. The cutting is commenced from an exposed edge, or a hole must first be burned from which to commence. After first applying the heating flame over the commencing point until the colour turns from blue to orange, the cutting lever is depressed and the flame moved very steadily at a uniform rate along the line to be cut.

DRIVING OF SHEET PILES

General.*

The piles should always be driven with the tongue leading, this applying equally to timber, concrete, or steel sheet piles. Sheet piles of any material tend to drive outward and also to creep in the direction in which the wall is being driven. This is usually prevented by pitching and driving to part penetration 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 leaning in the direction of driving has already taken place, it may be reduced and often corrected by pulling the heads of successive piles during driving, the

* See also page 85.

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.* 2, to correct the tendency to creep away sideways. Driving between temporary walings at the level of the pile heads when driven, as shown in *Fig.* 9, 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 guiding is possible, and as a makeshift, creeping forward or backward of timber or concrete sheet piles may be minimised by temporary steel clamps (*Fig.* 10) sliding against the last pile driven. Where one steel section is driven at a time, a clean female interlock is obtained by inserting a loose bolt or rivet at the bottom of the section before driving.

For driving sheeting of any material in cohesive soils, pile frames with drop hammers are generally used, the ratio of weight of hammer to weight of pile being a minimum of 1·5 and preferably just over 2·0 for steel sheet piles. For concrete

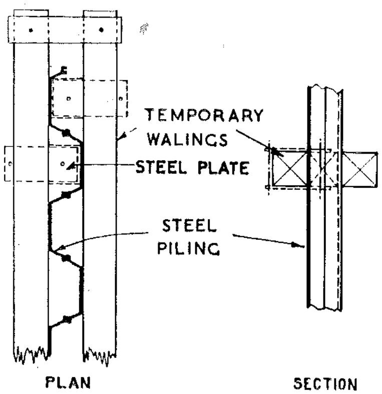


FIG. 9.—TEMPORARY WALINGS FOR THE ALIGNMENT OF STEEL SHEET PILES DURING DRIVING.

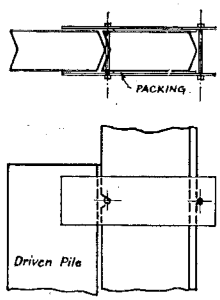


FIG. 10.—MAKE-SHIFT CLAMP FOR ALIGNING SHEET PILES WHEN WALINGS CANNOT BE USED.

sheet piles the ratio is best between 0·6 and 1·0 with the drop limited to about 3 ft. Single-acting steam hammers are an alternative, but they are more suited to heavy piles and are consequently seldom used for driving steel sheet piles.

For granular soils, including compact sand, the double-acting steam hammer is ideal because the rapid blows, by the resulting vibration in the sub-soil, will greatly assist in the penetration.

Continued blows on the pile cap when an obstruction or sudden resistance in 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 either shows that the pile has met 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.

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, which is possible with clays and with most granular soils.

Occasionally jetting pipes have been cast in the centre of reinforced concrete

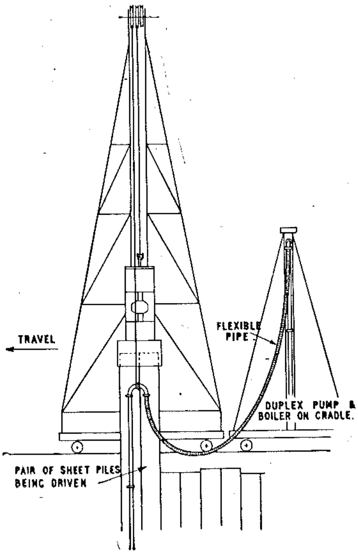


FIG. 11.—METHOD OF SUSPENDING JETTING PIPE.

piles, but this method is more expensive and not so efficient as a loose pipe. 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 soil and water around the pile. To enable this to be done, the jetting pipe or pipes are invariably slung from the pile driver or the derrick 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 $2\frac{1}{2}$-in. bore extra strong pipe, with the end reduced to a nozzle outlet not exceeding one-quarter the area of the pipe, say a $\frac{7}{8}$-in. or 1-in. nozzle for a 2-in. pipe. The water consumption with a 1-in. nozzle will 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 $7\frac{1}{2}$ in. by $4\frac{1}{2}$ 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 small, say in clay, otherwise there is a risk of delay due to shortage of steam supply. If the volume of water rising around the pile is at all deficient the jet pipes need to 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

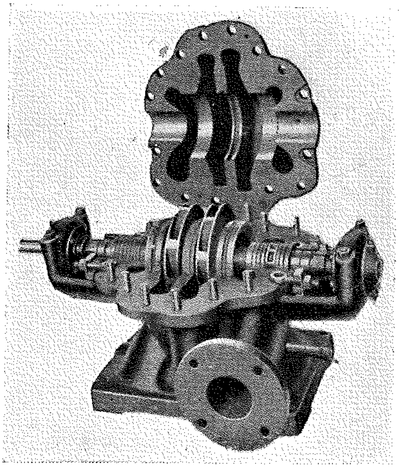


FIG. 12.—TWO-STAGE CENTRIFUGAL PUMP.

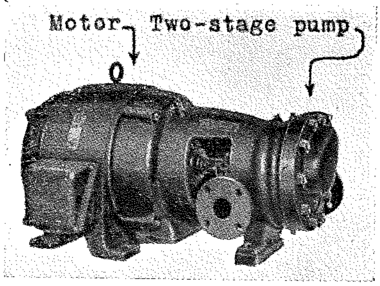


FIG. 13.—TWO-STAGE CENTRIFUGAL PUMP IN ONE UNIT WITH ELECTRIC MOTOR.

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.* 12 and, as the centrifugal pump is suitable for speeds of about 3,000 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.* 13). 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 $2\frac{1}{2}$-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 pile 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

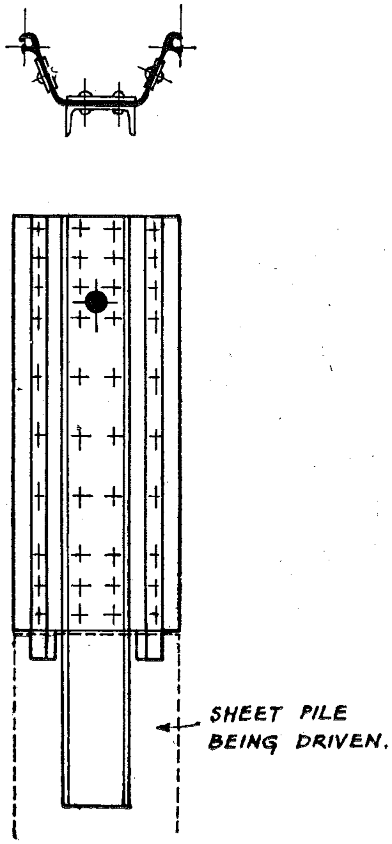


FIG. 14.—" FOLLOWER " FOR DRIVING STEEL SHEET PILES BELOW THE LEADERS.

and the top of the pile are 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.* 14, 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, and opinions differ on which is the better method. On the one hand the hammer loses efficiency by buoyancy, while on the other hand a follower both loses efficiency and needs extending leaders if there is to be good hope of the pile not wandering and being damaged by eccentric blows of the hammer.

Certain double-automatic hammers operate successfully at 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 Sheeting to Curves in Plan.

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 (as M107 and M108 of *Table* II), of 13 ft. $2\frac{1}{2}$ in. For types having less freedom in the interlocks, as sections IA, II, and III of *Table* II when in long lengths, the minimum diameter is about 60 ft., and reduces for shorter piles of, say, 16 ft. long to 30 ft. For full circles an even number of piles is, of course, necessary for types having interlocks on the alternate outer faces.

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, for example, of about 12,000 lb. per inch run in the case of types M107 and M108, so as to be suitable for use in cellular cofferdams dealt with later. On the other hand, with sections that have a high 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 for the stress being considered.

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.* 15). The interlock then serves as a lead for guiding the pile. A typical double-acting hammer is shown in *Fig.* 16. 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.

A pile frame is often used with a double-acting pile hammer (*Fig.* 17) instead of a drop hammer or monkey, since the pile frame greatly assists maintaining

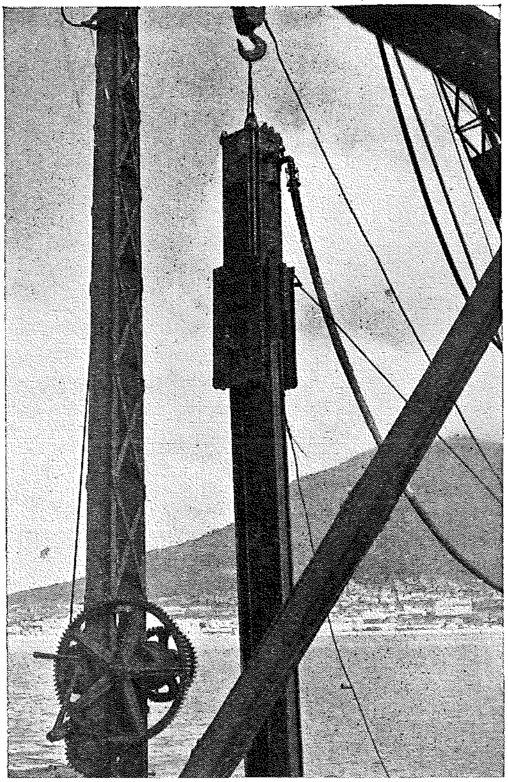


FIG. 15.—DOUBLE-ACTING STEAM HAMMER DRIVING STEEL SHEET PILES.

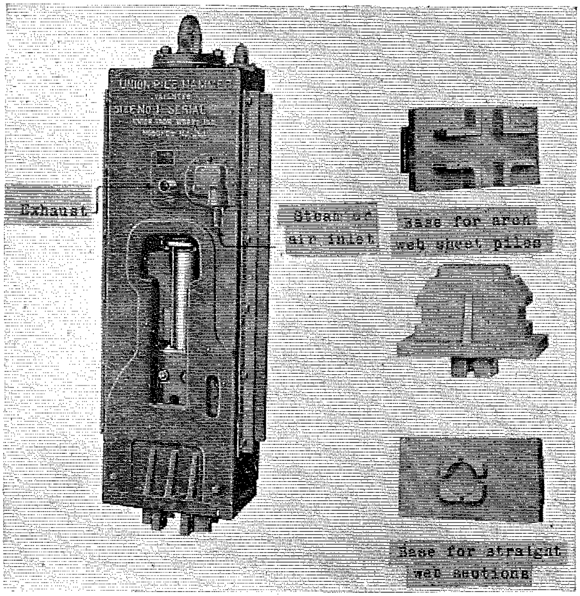


FIG. 16.—DOUBLE-ACTING PILE HAMMER WITH ALTERNATIVE BASES FOR DRIVING STEEL SHEET PILES.

alignment of the piling, while the rapid but light blows of the automatic double-acting hammer are better for penetrating permeable granular soils, particularly coarse sand and gravel. In this case the automatic hammer is held in the leaders by back or side guides. Double-acting hammers can be operated by steam or by compressed air; the correct lubricant must be used in each case. The hammer may be placed directly on the pile, or a driving head of the type shown in *Fig.* 18 to suit the pile section may be used. When driving steel sheeting without a pile frame, leg guides (*Fig.* 15) are commonly used in combination with temporary timber walings secured to temporary stakes or to the tops of the previously-driven piles.

FIG. 17.—DOUBLE-ACTING STEAM HAMMER USED WITH PILE FRAME.

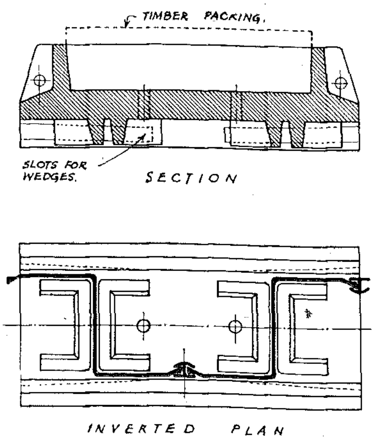


FIG. 18.—TYPICAL HELMET FOR DRIVING STEEL SHEET PILES IN PAIRS WITH A DROP HAMMER.

Extraction of Sheet Piles.

The successful operation of double-automatic hammers is dependent upon keeping the full weight of the hammer always on the pile, otherwise the full developed blow may be taken by the hammer frame with disastrous results. Some hammers are designed so that they will not start until resting fully on the pile, and this is a useful advantage.

In exceptional cases, say where the sheeting is being driven in inaccessible positions, or sometimes to avoid providing temporary stagings, use is made of hanging leaders suspended from a crane (*Fig.* 22).

Some types of steel sheeting piles are supplied already interlocked in pairs to save time in driving, and helmets as shown in *Fig.* 18 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 as single piles; it also makes it easier to guide the piles. Where steel sheet piling has to be driven in stiff cohesive soil (particularly thin web sections) the bottom edges of the piles are occasionally reinforced by bolted or welded steel strips so that the skin friction is reduced on the pile higher up.

If they are inverted and fitted with an extracting attachment (*Fig.* 19), double-acting hammers will withdraw steel piles as easily as they are driven; in fact, normally, more easily in the case of undamaged piles in non-cohesive soils. A different type of jaw is used for extracting concrete and timber piles.

When double-acting hammers are inverted and used for extracting piles, they are usually arranged so that steam is not admitted to the top of the cylinder, and sometimes also, particularly with the larger sizes of steam hammers, the exhaust from the low side is restricted so as to cushion the piston on its downward (idle) stroke when inverted. If a hole is provided in the web of steel sheet piles near the top, for example, as is usual in American practice, they can be extracted by an inverted automatic double-acting hammer using either a flexible yoke [*Fig.* 20 (*a*)] or a rigid yoke [*Fig.* 20 (*b*)], the latter consisting of easily-replaceable parts in case continual battering cause a fracture by crystallisation of the metal. The extracting gear should be properly annealed periodically to prevent crystallisation of the metal. Where large numbers of piles have to be withdrawn, an extractor (*Fig.* 21) is an alternative to the inverted double-acting hammers shown in *Figs.* 19 and 20.

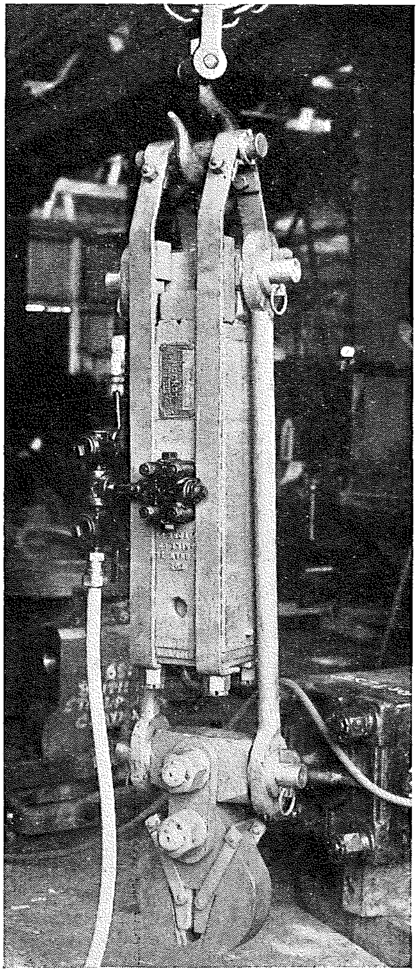
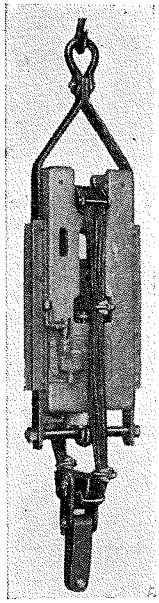


FIG. 19.—DOUBLE-ACTING STEAM HAMMER INVERTED FOR EXTRACTING SHEET PILES.

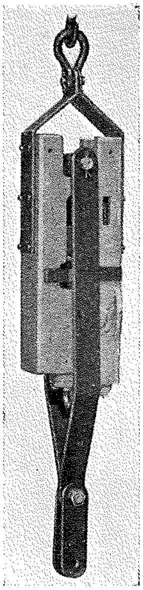
The extraction of undamaged steel sheet piles will normally be within the capabilities of an extractor suspended from a derrick,[71] even if the piling was driven some time previously. The exception would be for deep penetration in stiff clay. In this case, or if the piles are at all long and are damaged 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. 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 pile frame is generally used with a drop hammer, or sometimes with a single-acting steam hammer in the case of heavy concrete sheet piles.



(a)



(b)

FIG. 20.—DOUBLE-ACTING PILE HAMMERS INVERTED FOR EXTRACTING SHEET STEEL PILES.

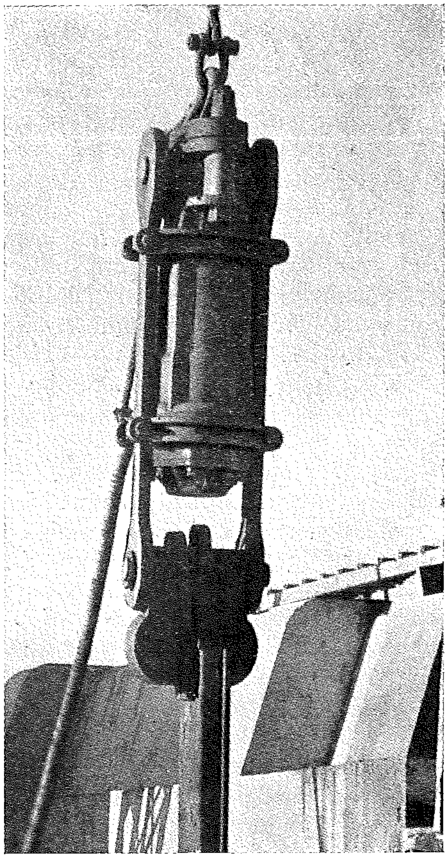


FIG. 21.—PILE EXTRACTOR.

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 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

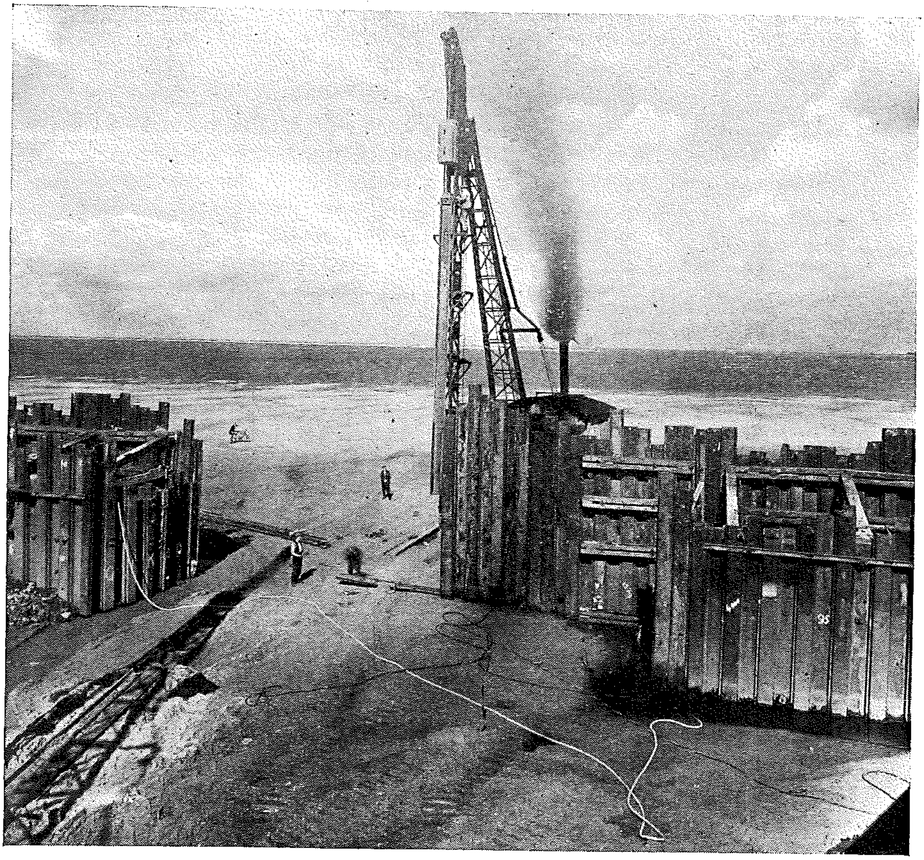


FIG. 22.—DRIVING STEEL SHEET PILES USING HANGING LEADERS.

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 main bars end flush with the head, and by proper, but not excessive, maturing of the pile before driving, say one month for rapid-hardening Portland cement and two to three months for ordinary Portland cement concrete. As a head packing, woven asbestos mats in several layers are likely eventually to replace the use of 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 formulæ.

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 and the ratio of weight of hammer to weight of timber pile should be as great as possible, preferably not less than $1\frac{1}{2}:1$.

Choice of Size of Hammer.

The most suitable weight of drop hammer to use for steel sheet piles is generally that nearest to $2\frac{1}{4}$ times the weight of the sheet pile being driven, a hammer just over this ratio being slightly preferable to one just under. This may be obtained from *Fig.* 23 by using the top scale for the length of the pile. The lower scale ($K = 1{\cdot}5$) corresponds to a drop hammer to pile weight ratio of

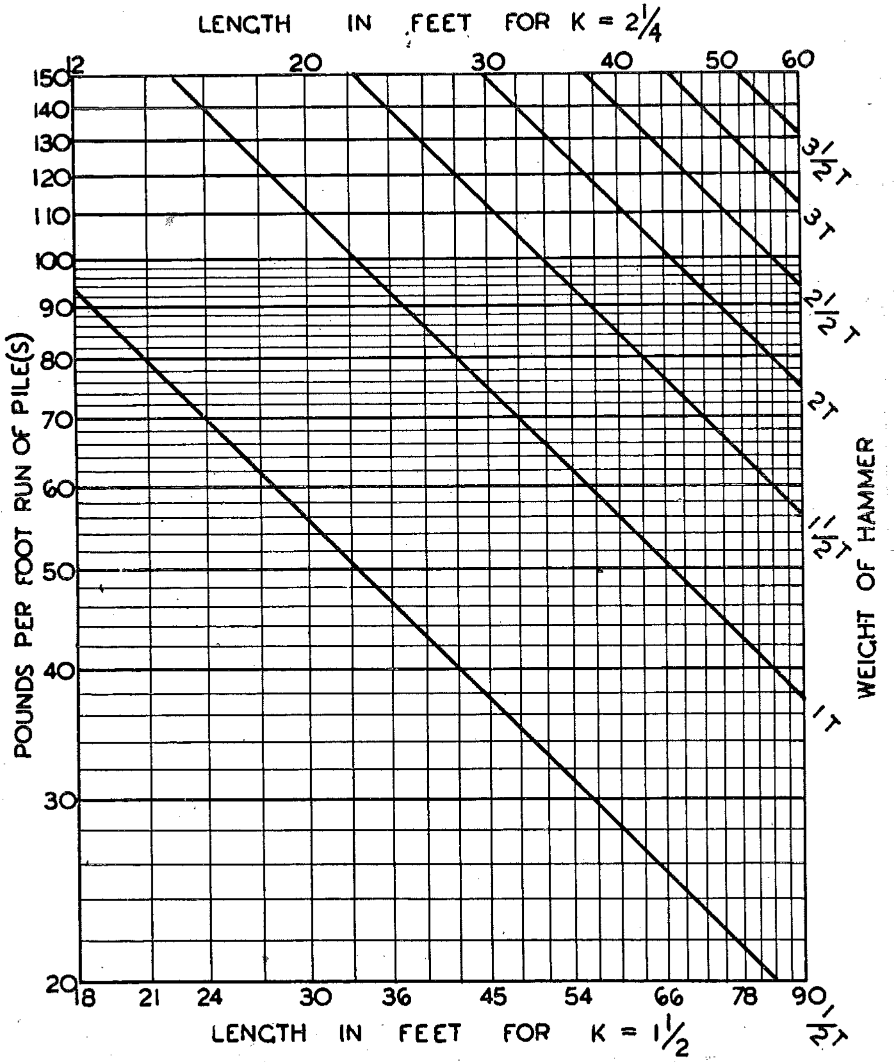


FIG. 23.—GUIDE TO CHOICE OF WEIGHT OF DROP HAMMER FOR DRIVING STEEL SHEET PILES.

1·5, and may be considered the lower limit for satisfactory driving in any but the softest soils. This graph may be used for timber piles since the same ratios of K will apply in most soils, but not for reinforced concrete piles where a much lower value of K is usual.

For double-acting hammers the ratio of the weight of the reciprocating parts to the whole weight of the hammer varies with different makes, while the type of soil has more effect on the best choice of size of hammer. As a generalisation, however, the ratio K (whole weight of double-acting hammer to weight of sheet pile, or pair of sheet piles) may be taken as the same as for drop hammers and the nearest hammer to $K = 2\frac{1}{4}$ used (*Fig.* 24). If this type of hammer is used in cohesive soils a higher value of K will generally be desirable, and a drop-hammer is more often used.

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.

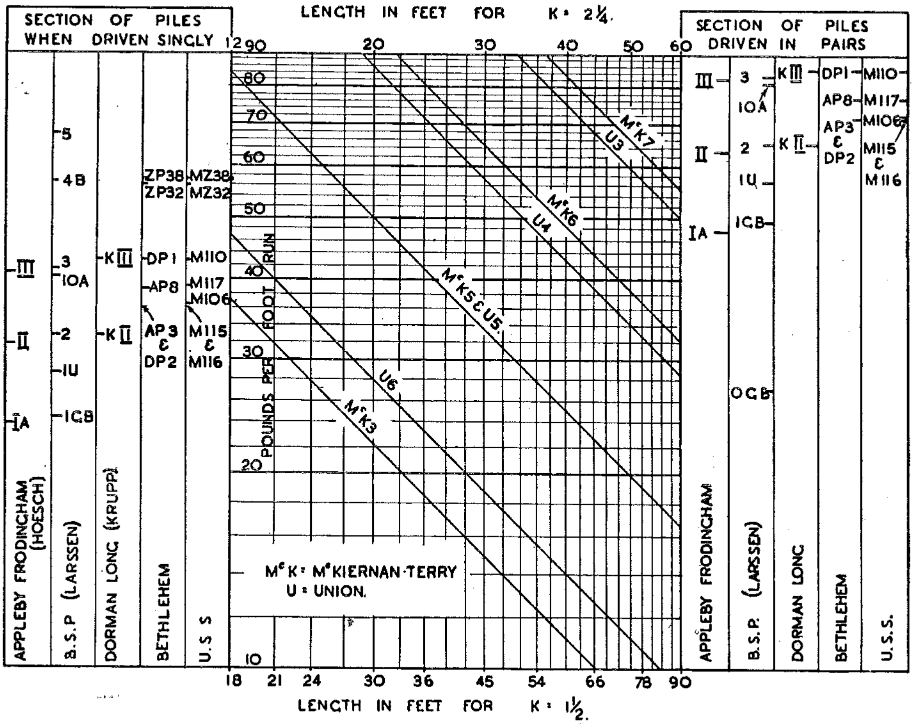


FIG. 24.—GUIDE TO CHOICE OF SIZE OF DOUBLE-ACTING PILE HAMMER FOR DRIVING STEEL SHEET PILES.

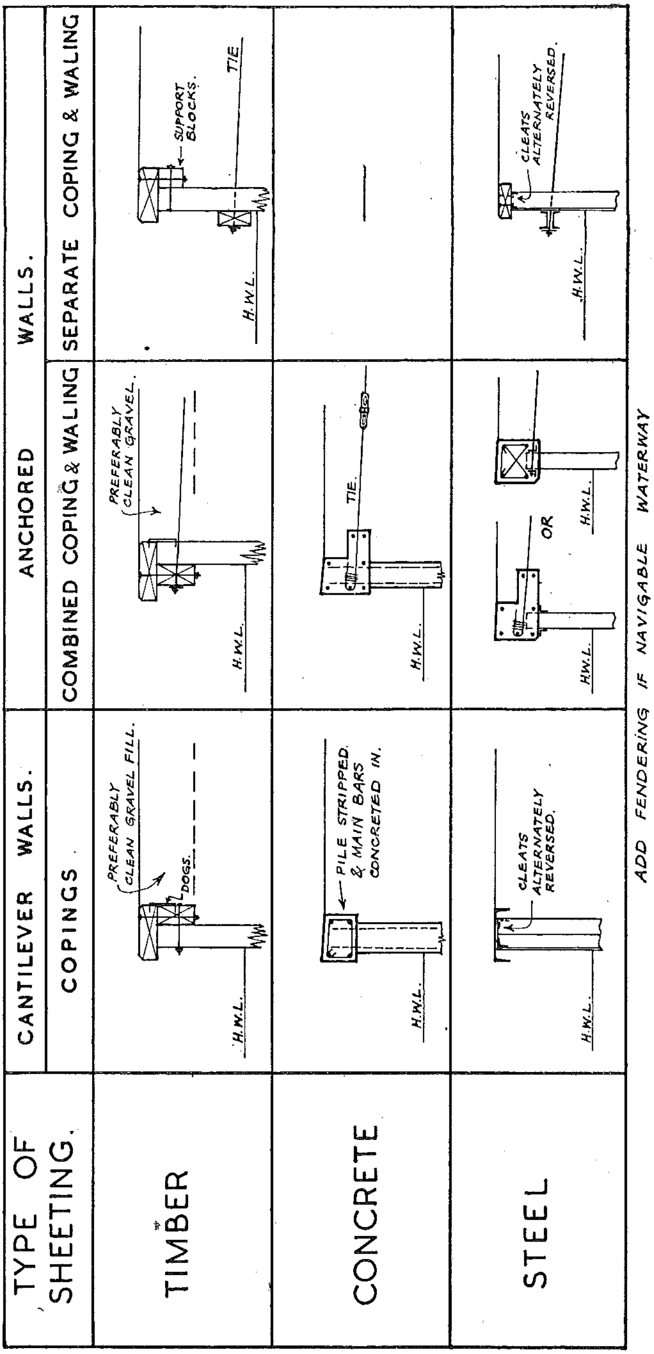


Fig. 25.—Types of Copings and Capping Beams.

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.* 25 are typical of river walls and may need increase or modification of detail if used for wharves. For tidal navigable waterways, if projections from the 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.

PART II

EARTH PRESSURES AND SHEET PILING

Although numerous theories have been put forward to demonstrate the form of the active and passive earth pressures supporting sheet-piling walls, these 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. In any case careful analyses of samples are necessary for close determination, and the characteristics of the soil may also vary fairly considerably in the length of the wall. For this reason straight line, or hydrostatic, pressure-distribution theories are generally used for both the active and passive pressures.

In *Fig.* 26 the pressure distribution generally assumed is shown at (*a*) and a pressure diagram which is in accordance with the probable action of sheet-piling walls in average soils is shown at (*b*). Subsequently it will be seen that the assumptions, although apparently fairly seriously in error, are in fact safe

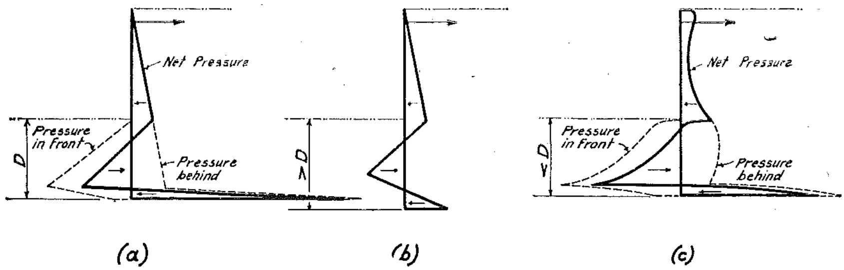


Fig. 26.—Distribution of Lateral Pressure.

for design, and facilitate the calculations which would otherwise be too involved for work other than that of exceptional importance.

The methods outlined here have the advantage of being adaptable to any current theory of active and passive earth pressures provided these have a hydrostatic variation, and any desired values of active and passive pressures may be used taking into account; also, if desired, the effect of friction. Active earth pressures may be based upon the formulæ of Rankine, Coulomb, Bell,[9] or Jenkin.[10, 11]

Recent research and many observations of the behaviour of earth-retaining structures have shown that the first three of these give a far from exact picture of earth pressure, unless they are qualified by additional conditions of the stiffness of the wall, the fixity of the base of the wall and the physical state of the soil. Thus, although the wedge theory presumes a straight-line plane of rupture, this normally takes the form of a curve. Also, experience has shown that this curve meets the surface within narrow limits notwithstanding the comparatively wide variations in the soil properties; thus Meem found it to lie very close to 0·4H, while Sir Benjamin Baker, Peckworth, Haines and others have found it to lie near 0·5H. 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 both the lateral pressure and the centre of pressure.

The formulæ of Rankine and Coulomb are generally satisfactory for use for

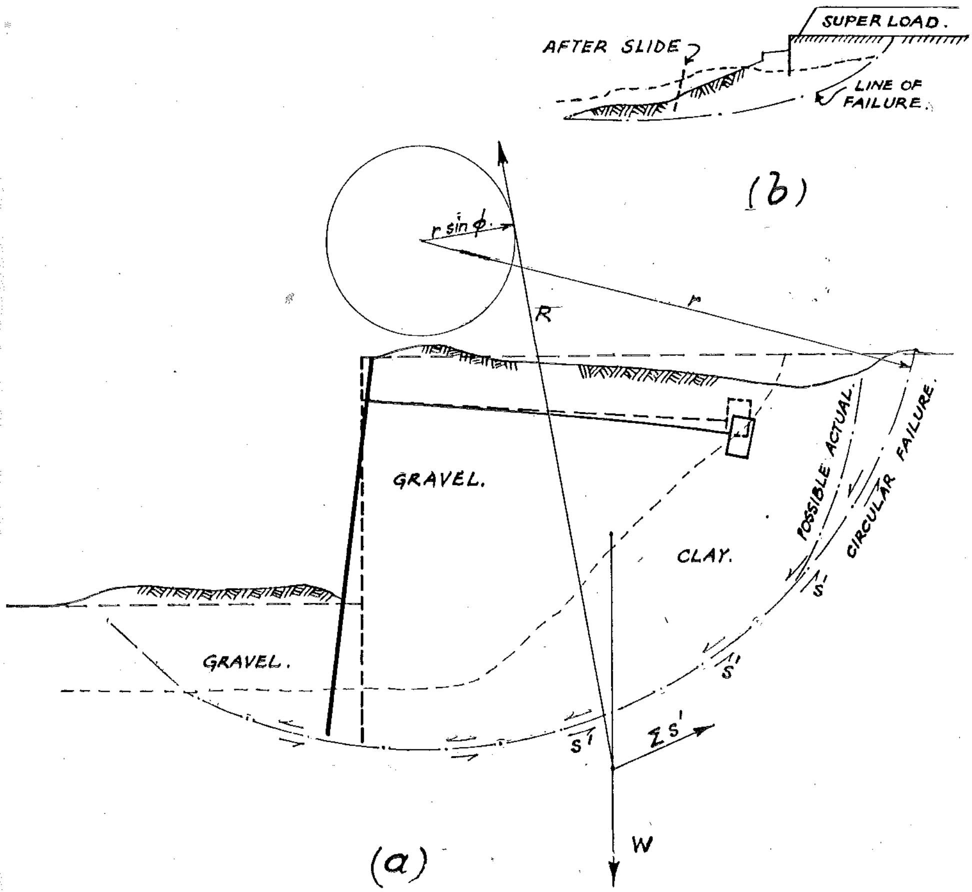


FIG. 27.—TYPICAL SUBSOIL FAILURE WITH COHESIVE SOIL OF LOW BEARING VALUE.

cohesive soils, provided the minimum angle of internal friction of the cohesive soil is taken as that of the soil at its highest expected water content and the assistance provided by cohesion is ignored. Professor Jenkin's formula, being dependent upon the properties of dilatation of granular soils discovered by Osborne Reynolds, cannot properly be used for cohesive soils.

Since cohesive soils normally have a relatively low angle of internal friction, and cohesion gives a constant addition to the shear strength, the type of failure of cohesive soils is frequently that shown by *Fig.* 27, 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 soil properties extending completely outside the areas of the wedge theories. From the loading and the soil properties the surface of sliding (shear) 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 by *Fig.* 27 (*b*).

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

$$P = \frac{wH^2}{2} \cdot \left(\frac{1 - \sin \phi}{1 + \sin \phi}\right) = \frac{wH^2}{2} \cdot \tan^2\left(45^\circ - \frac{\phi}{2}\right) \quad . \quad . \quad (1)$$

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

Coulomb's formula, although based upon slightly different assumptions, gives the same result in this case of level top surface.

Bell [9] 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

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

where C is the cohesion, that is the shear 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 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.

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 earth pressures for cohesionless granular soils and, assuming the small movement of the wall necessary to develop the assumptions, the active pressures found confirmed to a large extent the previous work of Resal. The active pressure factors are given in *Table* IV for comparison with Rankine's figures.

The centre of pressure taken at $\frac{H}{3}$ in using Rankine's method is, however, not justified by recent research, since it varies with the state of packing of the

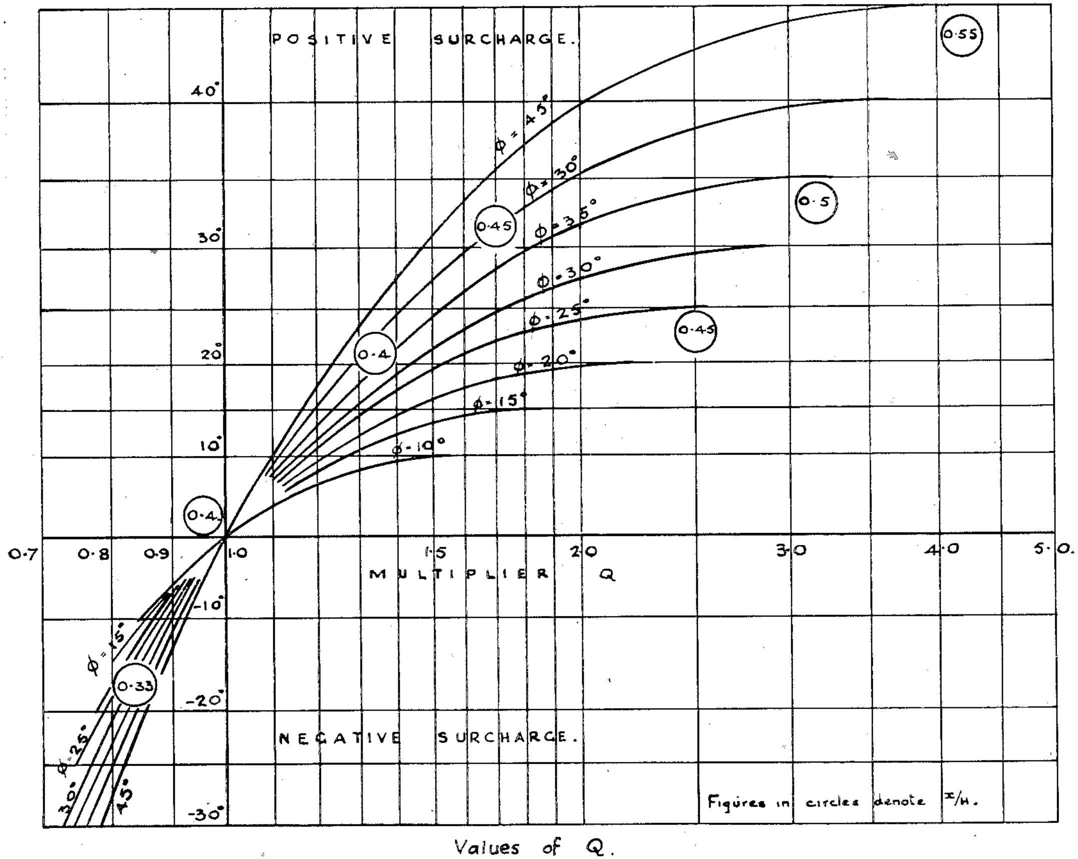
TABLE IV.

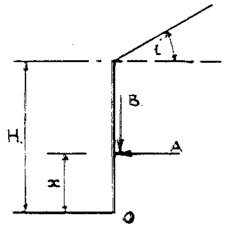
EARTH PRESSURE FORMULÆ AND VALUES.

		Rankine	Coulomb	Bell	Jenkin
	Type of soil	Granular	Granular	Cohesive	Cohesionless granular
Level top surface	Active horizontal pressure at a depth h with level top surface	$wh.\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$	$wh.\tan^2\left(45-\frac{\phi}{2}\right)$	$wh.\tan^2\left(45-\frac{\phi}{2}\right)$ $-2C.\tan\left(45-\frac{\phi}{2}\right)$	Given in table on *Fig.* 28
	Maximum passive horizontal resistance at a depth d	$wd.\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$	$wd.\tan^2\left(45+\frac{\phi}{2}\right)$	$wd.\tan^2\left(45+\frac{\phi}{2}\right)$ $+2C.\tan\left(45+\frac{\phi}{2}\right)$	—
	Maximum passive vertical resistance to downward pressure alongside at a depth d	$wd.\left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$	$wd.\tan^4\left(45+\frac{\phi}{2}\right)$	$wd.\tan^4\left(45+\frac{\phi}{2}\right)$ $+2C\tan^3\left(45+\frac{\phi}{2}\right)$ $+2C\tan\left(45+\frac{\phi}{2}\right)$	—
Surface inclined at an angle i	Active pressure at a depth h with surcharge angle $i=\phi$	$whQ\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ (see *Fig.* 28)	Graphical method usually used	—	Obtain from graph on *Fig.* 28
	Maximum passive resistance at a depth d	Use graphical method, say as *Fig.* 32	Ditto	—	—

Angle of internal friction ϕ (deg.)	Active horizontal pressure (lb. per sq. ft.) at a depth h for a soil (w = 100 lb. per cu. ft.) and level top surface		Maximum passive horizontal resistance (lb. per sq. ft.) to downward pressure at a depth d for a soil (w = 100 lb. per cu. ft.) and level top surface
	Rankine and Coulomb	Jenkin	Rankine and Coulomb
5	83·8	78·7	119
10	70·2	62·5	142
15	58·8	50·0	170
20	48·9	40·1	204
25	40·5	32·2	247
30	33·3	25·7	300
35	27·0	20·5	370
40	21·7	16·1	460
45	17·2	12·5	580

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.





ϕ	$A'_{i=0}$	$B'_{i=0}$
45°	0·125	0·125
40°	0·161	0·135
35°	0·205	0·143
30°	0·257	0·149
25°	0·322	0·150
20°	0·401	0·146
15°	0·500	0·134
10°	0·625.	0·110

$Q = \frac{A}{A'_{i=0}}$

The Horizontal Component P_a, of p is then

$P_a = A'Q\,wH$

$M_o = \simeq A'Q\frac{wH^2}{2}$

FIG. 28.—ACTIVE EARTH PRESSURE FOR COHESIONLESS GRANULAR SOILS WITH SURCHARGE.

soil and slight movement of the wall. Professor Jenkin recommends Weyrauch's proposal that the centre of pressure should be taken to act at $0{\cdot}4H$ for flat top surfaces and for surcharge angles between -10 deg. and $+25$ deg. Burmister has also suggested $0{\cdot}4H$ for the average centre of pressure. With a positive surcharge angle i approaching ϕ, the centre of pressure is higher and reaches $0{\cdot}55H$ at $i = \phi$. The variation of the centre of pressure with tilting of the wall is reproduced in *Fig.* 29 after Terzaghi.[12]

In addition, the soil arches across deflected wall surfaces and concentrates, in the case of anchored sheeting walls, near the top 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-piled walls, and a slight simplification of that method will be given later. Before doing so, however, it is necessary to note that a combination of values from different sources may result in ignoring qualifications to their use. Thus, assumptions for the active pressure found to lead to a safe design may be stated:

(*a*) Angle of internal friction taken as lowest likely during the life of the structure, which is usually the value when the soil is saturated.

(*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 no water is in front of the wall, 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 $0{\cdot}4H$, 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 by *Table* IV and at the side of *Fig.* 28.

For rigid walls which become loaded by pumped filling, the temporary added hydrostatic pressure 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-piled 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 values by Rankine's formula for positive surcharge are known to be excessive. The centre of pressure should then be taken as $0{\cdot}4H$, or higher for positive surcharge, as the acceptance of yield to reduce the active pressure is seen by *Fig.* 29 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 Rebhann'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 giving directly the coefficients for the active pressure in horizontal and vertical directions.

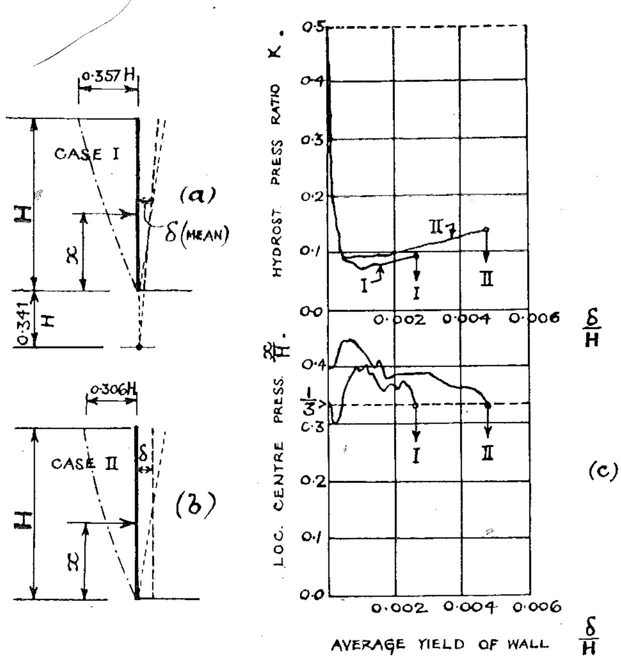


FIG. 29.—MOVEMENT OF CENTRE OF PRESSURE WITH SLIGHT TILTING AND FORWARD MOVEMENT OF A WALL (AFTER TERZAGHI).

These coefficients can be used in conjunction with Rankine's formula for active pressure if the coefficients are multiplied by the factor

$$\frac{p_a \text{ (by Rankine)}}{p_a \text{ (by Jenkin)}}, \text{ both taken for } i = 0$$

and for the correct value of ϕ.

A combination of assumptions for the active pressure limited to granular soils which is more in keeping with recent research is therefore

(*a*) Angle of internal friction taken as the lowest for the material, usually the value when saturated and between 7 deg. and 10 deg. less than when dry.

(*b*) Active pressure by Rankine's formula if the surface is level, or proportional to Professor Jenkin's values as just described if the surface slopes.

(*c*) Centre of pressure $0{\cdot}4H$ for all normal types of sheet walls and surcharge angles between -10 deg. and $+25$ deg.

(*d*) Reduction values as in *Fig.* 30 applied to moments in sheeting. (No reduction in moment for cantilever walls or to the total active pressure in either case.)

(*e*) Friction ignored for active pressure.

(*f*) Cohesion entirely ignored.

It should be mentioned, however, that if the soil is not known to be cohesionless when wet and dry Professor Jenkin's tables are not applicable.

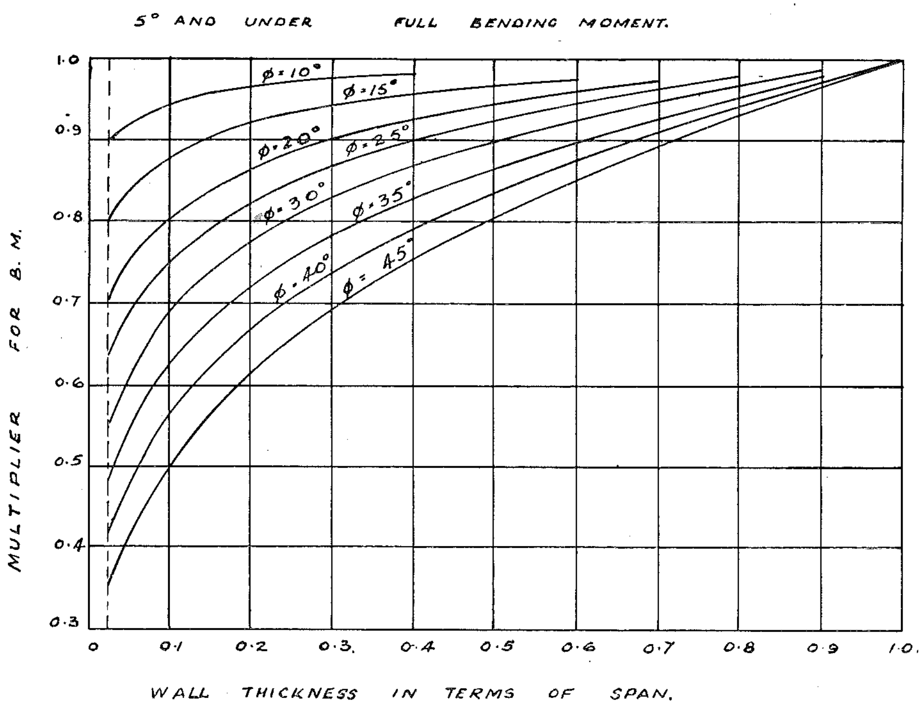


FIG. 30.—MOMENT REDUCTION FACTORS FOR SHEET WALLS (AFTER MR. R. N. STROYER).

It is not from choice that a sheet-piled 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 shear 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-piled 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 ϕ 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 by *Table* V 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 varying from 0·2 ton per square foot for very soft clay to 1·6 tons per square foot for very stiff boulder clay; the values for ϕ are 0 deg. and 16 deg. respectively. It is recommended, however, that the actual properties of the cohesive soil concerned in the proposed construction should be ascertained if it is considered necessary to take account of cohesive properties in the design.

TABLE V.

VALUES OF COEFFICIENTS IN BELL'S FORMULA FOR VARIOUS VALUES OF ANGLE OF INTERNAL FRICTION.

ϕ (deg.)	$\tan^4\left(45+\frac{\phi}{2}\right)$	$\tan^3\left(45+\frac{\phi}{2}\right)$	$\tan^2\left(45+\frac{\phi}{2}\right)$	$\tan\left(45+\frac{\phi}{2}\right)$	$\tan^2\left(45-\frac{\phi}{2}\right)$	$\tan\left(45-\frac{\phi}{2}\right)$
1	1·072	1·054	1·035	1·018	0·9657	0·9827
2	1·150	1·110	1·072	1·035	0·9326	0·9657
3	1·233	1·170	1·111	1·054	0·8905	0·9490
5	1·416	1·300	1·191	1·091	0·8397	0·9163
7	1·630	1·444	1·278	1·130	0·7828	0·8847
10	2·017	1·693	1·420	1·192	0·7041	0·8391
15	2·885	2·213	1·698	1·303	0·5875	0·7673
20	4·160	2·913	2·040	1·428	0·4903	0·7002

Superimposed Loading on Surface.

When, as usual, there is a superimposed load on the surface, this may be treated as equivalent to an increased depth of soil; thus 240 lb. per square foot superimposed loading is represented, for soil weighing 120 lb. per cubic foot, by calculating the pressures as if H (see *Figs.* 33 and 38) were increased by $\frac{240}{120} = 2$ ft.

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 wh 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 ϕ' is the angle of internal friction in water, then at a depth h below a common water and earth surface

$$p_a = 62{\cdot}5h + \left[w_e - \left(\frac{100 - v}{100}\right)62{\cdot}5\right]\left(\frac{1 - \sin\phi'}{1 + \sin\phi'}\right)h \quad . \quad . \quad (3)$$

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 specific gravity of the soil constituents seldom varies much from that of quartz (2·7), while for sedimentary granular soils in their natural condition the assumption of a porosity of 40 per cent. is a rough but reasonable approximation.

Density of Soil and Angle of Internal Friction.

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 (ϕ), 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* VI gives typical values of density and angles of repose for a variety of soils.

TABLE VI.

DENSITY AND ANGLE OF REPOSE OF SOILS.

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

Material	Density (lb.) when dry		Angle of repose	
	Loose	Compacted or natural deposit	Dry (or wet in still water)	Wet in slow-flowing water
Coarse sand (well graded)	100 (40)	115 (31)	32–38	20
Fine sand	90 (46)	105 (37)	30–35	15
Gravel	110 (34)	120 (28)	35	25
Shingle	95 (43)	100 (40)	40	40
Brick hardcore	80	110	40–45	40–45
Quarry waste	90–100	100–120	40–50	45–50
Broken stone	90 (46)	110 (34)	45	40
River mud	—	90	15–20	5–15
Sandy clay	—	110	5–15*	—
Soft clay	—	95	5*	—
Medium clay	—	105	10*	—
Very stiff clay	—	115	15*	—

* Angle of internal friction. Cohesion to be allowed for separately, say, as 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 ϕ, 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 ϕ. 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 the driving of piles. In both cases it may be expected the lateral pressure below the water level will not be less than that of the same soil above water level.

Passive Earth Resistance.

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

Bell's formula is seldom used because the foot of the wall is frequently 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 in the other direction, but 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. In fact in America the actual resistance to forward movement of the wall at the foot is frequently assumed to be twice that given by Rankine's formula. On the other hand, by the Danish rules greater emphasis is placed on substantial depth of driving by requiring a penetration such that the horizontal component of the passive resistance corresponds to twice the active force balanced by the earth resistance at the foot of the wall, and gives provisos elaborating this where the piles are spaced apart.

The apparent contradictions in these two methods are not as great as they would seem. If the assistance of friction to the passive resistance of the soil is not taken into account, it will be seen from *Fig.* 31 that a substantial increase in the passive resistance has been ignored. It will be noticed, however, from *Fig.* 31 that the equivalent added passive resistance due to friction is largely dependent on the angle of internal friction of the soil. The angle of friction between sheet piling and the soil cannot exceed the smaller of the two values: ϕ, the angle of internal friction of the soil, or $\tan^{-1}\mu$, the maximum angle of friction between the sheet piling and the soil. For wet materials the writer suggests $\tan^{-1} 0{\cdot}3$ for steel against sand, and $\tan^{-1} 0{\cdot}4$ for smooth concrete against sand or gravel.

In the Danish rules the limitation is an assumption that the direction of the passive earth pressure may be inclined upward at an angle of $\frac{3}{4}\phi$ with the normal of the wall, and cohesion ignored as already suggested. The writer's method is possibly better, however, as it prevents unintentionally taking $\frac{3}{4}\phi$ as a greater angle than the angle of friction between the piles and the soil.

The results by any method will depend largely upon the correct estimation of ϕ for the soil in front of the foot of the wall, and cohesion should not be depended

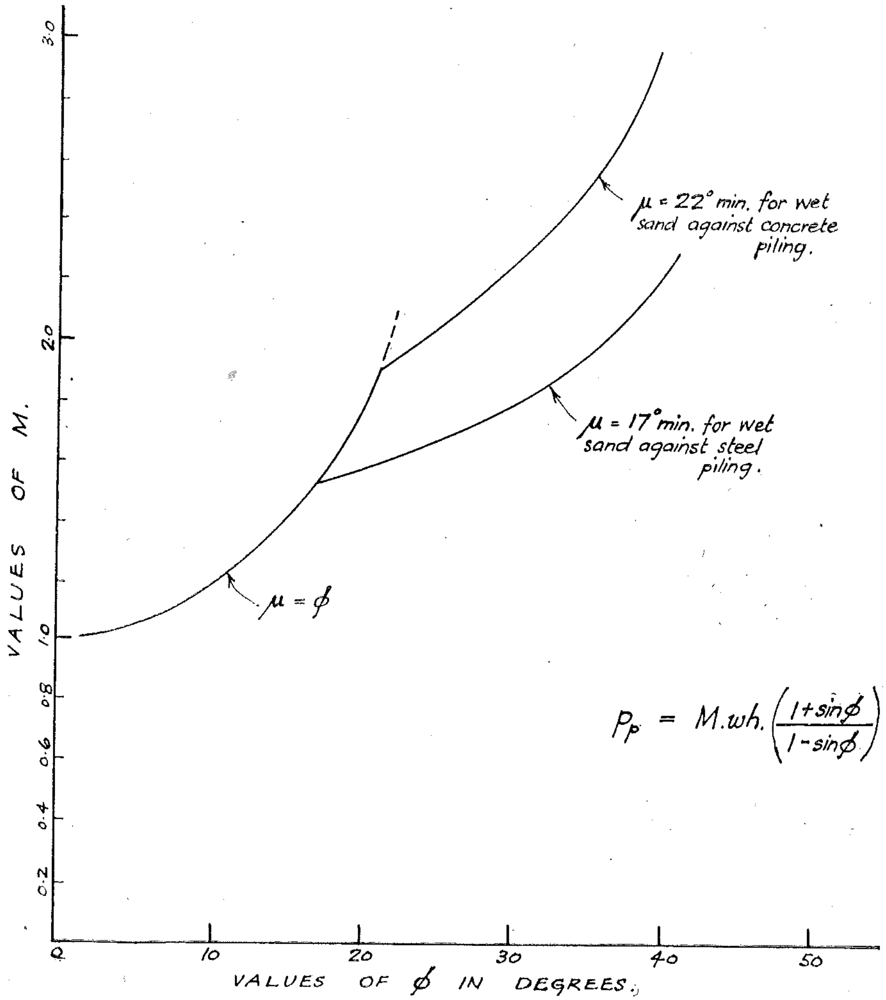


FIG. 31.—EFFECT OF FRICTION IN INCREASING THE PASSIVE RESISTANCE OF SOIL.

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 obtained from *Fig.* 31 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 forward movement of the

foot of the wall, ignoring any assistance from cohesion. For anchored walls use the same increase in penetration if driven only to obtain simple support at the bottom, graduating down to an addition to the net penetration of 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.

Passive Resistance with Negative Surcharge Slope.

A method of obtaining passive earth pressure when the surface of the soil recedes at a negative angle i from the face of the sheet piling has been deduced graphically by Professor Andersen [13] which is an extension of Poncelet's con-

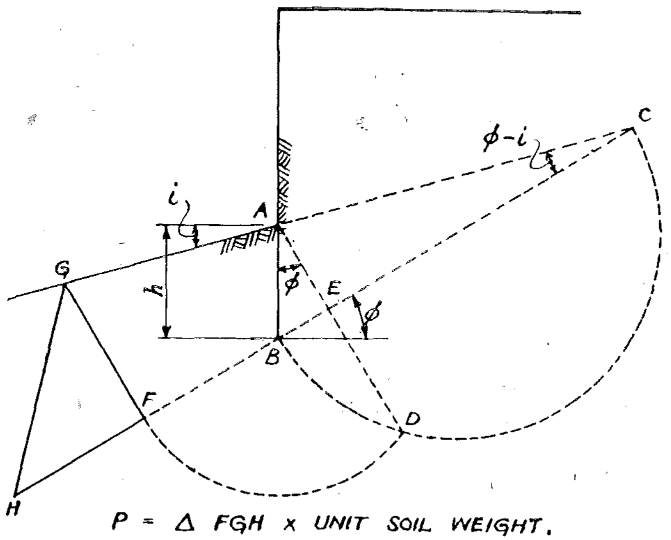


FIG. 32.—GRAPHICAL METHOD FOR PASSIVE RESISTANCE FOR NEGATIVE SURCHARGE OF SLOPE OF CHANNEL BED.

struction and is applicable to cohesionless soils. Taking the passive pressure from Coulomb, the graphical construction as shown by *Fig.* 32 is as follows.

The active pressure of a cohesionless soil behind a sheet-piled wall can be expressed graphically as the area of a right triangle multiplied by the unit weight of the soil. As will be shown, this determination can be extended to include passive pressures in front of walls which, according to Coulomb, can be expressed as

$$P = \frac{w}{2}\left[\frac{h\cos\phi}{1 - \sqrt{\sin\phi\,(\sin\phi - \cos\phi\tan i)}}\right]^2 \quad . \quad . \quad . \quad (4)$$

where w = unit weight of soil; ϕ = angle of internal friction; and
i = angle between the horizontal and the surface.

The graphical construction (*Fig.* 32) is as follows. Locate the point of intersection C between the extension of the front bank surface AG and a line through the bottom B of the sheet piling, and making an angle with the horizontal equal to the angle of internal friction ϕ. 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 erect GF normal to BC. If FH is made equal to FG, then the area of triangle GFH multiplied by the soil weight w will be equal to the passive pressure P.

The distance between F and C can be expressed as the sum of FB and BC, that is, as the sum of BD and BC. Thus

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

$$= h\,\frac{1 + \sqrt{\sin\phi\,(\sin\phi - \cos\phi\tan i)}}{\sin\phi - \cos\phi\tan i} \qquad . \quad . \quad . \quad (5)$$

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

$$FG = h\frac{1 + \sqrt{\sin\phi\,(\sin\phi - \cos\phi\tan i)}}{\cos\phi + \sin\phi\tan i} \qquad . \quad . \quad . \quad (6)$$

If the numerator and denominator in (6) are multiplied by

$$1 - \sqrt{\sin\phi\,(\sin\phi - \cos\phi\tan i)}$$

this expression becomes

$$FG = \frac{h\cos\phi}{1 - \sqrt{(\sin\phi - \cos\phi\tan i)\sin\phi}} \qquad . \quad . \quad . \quad (7)$$

which is the term appearing between brackets in (4).

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

When the channel bed is not horizontal, but slopes downward away from the wharf 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 wharf 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' may be used according to the following table, where D is the calculated or the tabulated penetration for the actual height for a horizontal bed.

	Values of H'	
	Coarse sand or gravel subsoil $\phi = 30$ deg.	Fine sand subsoil $\phi = 25$ deg.
5 deg.	$H + 0{\cdot}11D$	$H + 0{\cdot}13D$
10 ,,	$H + 0{\cdot}22D$	$H + 0{\cdot}27D$
15 ,,	$H + 0{\cdot}33D$	$H + 0{\cdot}41D$
20 ,,	$H + 0{\cdot}45D$	$H + 0{\cdot}55D$
25 ,,	$H + 0{\cdot}57D$	$H + 0{\cdot}71D$
30 ,,	$H + 0{\cdot}71D$	—

Friction.

Friction between the soil and sheet-piled walls does not have great effect in reducing the resulting active pressure behind the wall but, as just mentioned, has a fairly large effect in assisting the passive resistance of the soil. The effect of friction on the active pressure will be ignored but its effect on the passive resistance will be taken into account. The effect of friction between the sheet piling and the retained earth must be limited to the coefficient of friction that will obtain under the most unfavourable conditions, which applies when the soil is wet.

The friction between sheet steel piling and retained earth may be assumed to have a value of 0·3 to 0·6 of the active horizontal earth pressure, the lower value applying to wet sand, and the higher to dry sand, while, from some tests by the writer of the friction between sand and fairly smooth concrete, for wet sand the coefficient cannot safely be taken higher than 0·4. While these values may appear low, the desirability of conservative estimation will be apparent; and Brennecke and Lohmeyer, it may be noted, give a value of 0·3 for wet sand against smooth masonry.

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 2 is frequently taken, but in the following examples the Rankine value is only increased for friction as shown by *Fig.* 31, and no other multiplier has been used. On the other hand some increase over the calculated penetration is necessary if the Rankine value is considered the measure of the actual passive resistance.

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

$$\left[wh\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)\right] \qquad . \quad . \quad . \quad . \quad . \quad (8)$$

by 1·5 for soils in which $\phi > 20$ deg., and having regard to current theories this is conservative.

The graph (*Fig.* 31) has been used in the examples which follow and is slightly preferable to the more usual method of assuming the friction against the piling to be $\frac{3}{4}\phi$. Also ϕ represents whichever internal angle of friction of the soil is applicable, for instance, 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, will greatly affect this angle.

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.

Cantilever Sheet Walls.

Sheet-piled walls driven to sufficient penetration to retain earth, without the assistance of support at the top, form the simplest case. However, as the necessary depth of penetration is fairly large compared with the height of the retained earth, their use is mostly restricted to cases where the height of retained earth is small or local circumstances prevent anchorage of the top of the wall.

The following method of calculation may be considered to be a 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.* 26 (a) and 33.

Equating horizontal forces and multiplying by 2,

$$p_a(H + D)^2 - p_pD^2 + (p_p - p_a)(H + 2D)X = 0 \qquad (9)$$

$$X = \frac{p_pD^2 - p_a(H + D)^2}{(p_p - p_a)(H + 2D)} \qquad (10)$$

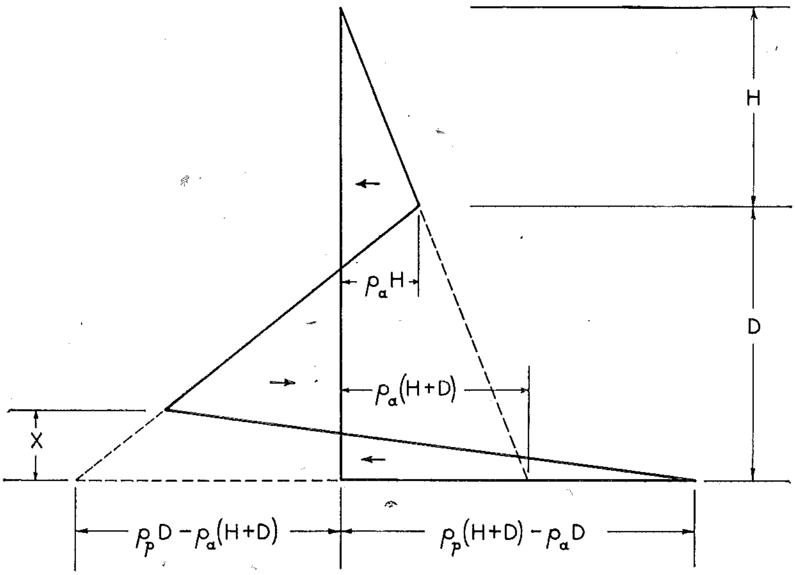


FIG. 33.—CANTILEVER SHEET RETAINING WALLS.

Equating moments of these forces about the foot of the sheeting and multiplying by 6,

$$p_a(H + D)^3 - p_pD^3 + (p_p - p_a)(H + 2D)X^2 = 0 \qquad (11)$$

To simplify, use a factor F such that $D = FH$.

Substituting for D in equation (10),

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

Substituting for D and X in (11),

$$p_a(H + FH)^3 - p_pF^3H^3 + (p_p - p_a)(H + 2FH)\left\{\frac{p_pF^2H^2 - p_a(H + FH)^2}{(p_p - p_a)(H + 2FH)}\right\}^2 = 0.$$

Simplifying,

$$p_a(H + FH)^3 - p_pF^3H^3 + \frac{\{p_pF^2H^2 - p_a(H + FH)^2\}^2}{(p_p - p_a)(H + 2FH)} = 0.$$

Extending,

$$p_aH^3(1 + 3F + 3F^2 + F^3) - p_pH^3F^3 + \frac{\{p_pH^2F^2 - p_aH^2(1 + 2F + F^2)\}^2}{(p_p - p_a)H(1 + 2F)} = 0.$$

Multiplying across and dividing by H^4,

$$p_a(p_p - p_a)(1 + 5F + 9F^2 + 7F^3 + 2F^4) - p_p(p_p - p_a)(F^3 + 2F^4)$$
$$+ p_p^2F^4 - 2p_pp_a(F^2 + 2F^3 + F^4) + p_a^2(1 + 4F + 6F^2 + 4F^3 + F^4) = 0 \quad (12)$$

Substituting the values of p_p and p_a gives a simple expression in F^4, from which F is obtained and therefore D.

EXAMPLE.—

If $p_a = 30$ and $p_p = 400$,

then

$$(p_p - p_a) = 370$$
$$p_a\ (p_p - p_a) = 11{,}100$$
$$p_p\ (p_p - p_a) = 148{,}000$$
$$p_a^2 = 900$$
$$p_p^2 = 160{,}000$$
$$2p_p \cdot p_a = 24{,}000$$

and equation (12) becomes

$$\begin{array}{rrrrr}
+\ 11{,}100 & +\ 55{,}500F & +\ 99{,}900F^2 & +\ 77{,}700F^3 & +\ 22{,}200F^4 \\
 & & & -\ 148{,}000F^3 & -\ 296{,}000F^4 \\
 & & & & +\ 160{,}000F^4 \\
 & & -\ 24{,}000F^2 & -\ 48{,}000F^3 & -\ 24{,}000F^4 \\
+\ 900 & +\ 3{,}600F & +\ 5{,}400F^2 & +\ 3{,}600F^3 & +\ 900F^4 \\
\hline
12{,}000 & +\ 59{,}100F & +\ 81{,}300F^2 & -\ 114{,}700F^3 & -\ 136{,}900F^4 = 0
\end{array} \quad (13)$$

Dividing by 136,900 throughout and changing signs,

$$F^4 + 0{\cdot}837F^3 - 0{\cdot}593F^2 - 0{\cdot}432F - 0{\cdot}088 = 0 \quad (14)$$

Differentiating,

$$4F^3 + 2{\cdot}51F^2 - 1{\cdot}19F - 0{\cdot}43 \quad (15)$$

For a first approximation try $F = 1$.

Substituting in (14),

$$1{\cdot}000 + 0{\cdot}837 - 0{\cdot}593 - 0{\cdot}432 - 0{\cdot}088 = +\ 0{\cdot}724 \quad (16)$$

and in (15)

$$4{\cdot}00 + 2{\cdot}51 - 1{\cdot}19 - 0{\cdot}43 = +\ 4{\cdot}89 \quad (17)$$

The approximate correction of this trial value is obtained by dividing the remainder in (16) by the remainder in (17) and changing the sign. Thus

$$F = \text{approximately } 1{\cdot}00 - \left(\frac{0{\cdot}724}{4{\cdot}89} = 0{\cdot}148\right) = 0{\cdot}852.$$

Again substituting in (14),

$$0{\cdot}527 + 0{\cdot}518 - 0{\cdot}431 - 0{\cdot}367 - 0{\cdot}088 = +\ 0{\cdot}159,$$

and in (15)

$$2{\cdot}47 + 1{\cdot}82 - 1{\cdot}01 - 0{\cdot}43 = 2{\cdot}85.$$

$$\therefore\ F = \text{approximately } 0{\cdot}852 - \left(\frac{0{\cdot}159}{2{\cdot}85} = 0{\cdot}056\right) = 0{\cdot}796.$$

It will be seen by inspection that it is sufficiently accurate for practical purposes to assume that $F = 0{\cdot}79$; if the process is repeated it will be found that $F = 0{\cdot}789$.

The penetration has been multiplied by $\sqrt{2}$, so as to provide a factor of safety of about 2 against the wall tilting forward due to over-estimating the characteristics of the soil, and the resulting increased penetration is that which has been plotted in *Fig.* 34. It should be noted that, in calculating the penetration for this graph, the value of p_p used already allows for the assistance of friction according to *Fig.* 31; for example, the values of p_p in the equations are M times $wh\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$.

For comparison only, curves obtained by Pennoyer [14] are given for the two cases of angle of friction of the soil against the sheeting of 0 and $0{\cdot}6\phi$. Although

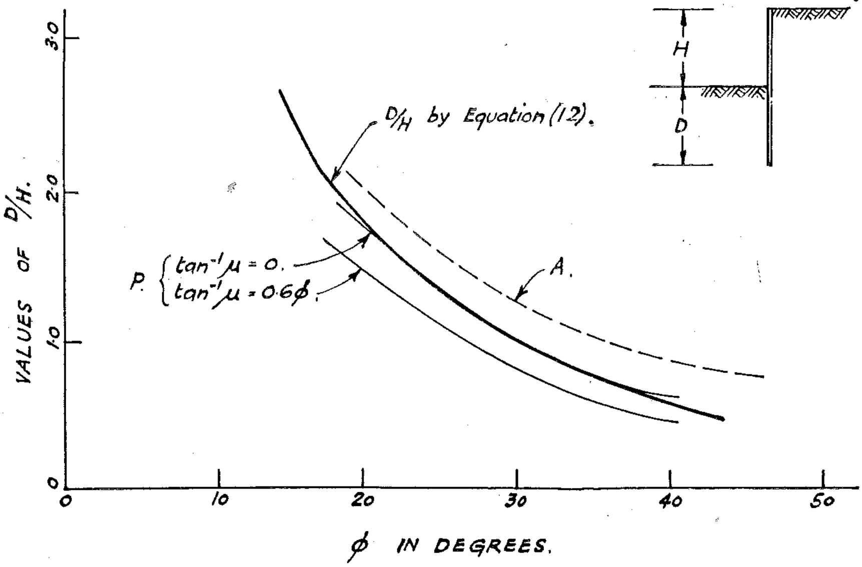


FIG. 34.—PENETRATION OF SHEET PILES FOR CANTILEVER WALLS.

not stated by Pennoyer it would appear that the Rankine passive resistance has been multiplied by two in obtaining these values of penetration. The dotted line shows values of penetration frequently given in handbooks, ignoring friction. Obviously the factor of safety against the foot of a cantilever 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 ground level and calculate the equivalent height h such that $p_a h$ is equal to the pressure at lower ground level produced by the actual loading. Then, provided that the soil below lower ground level is uniform, the method of calculation already given may be adapted by substituting in (9) for $p_a(H + D)^2$ the expression $2\left(P_1 + p_a hD + \frac{p_a D^2}{2}\right)$, where P_1 is the total active pressure above lower ground level, and making corresponding changes throughout, noting particularly the use of h instead of H.

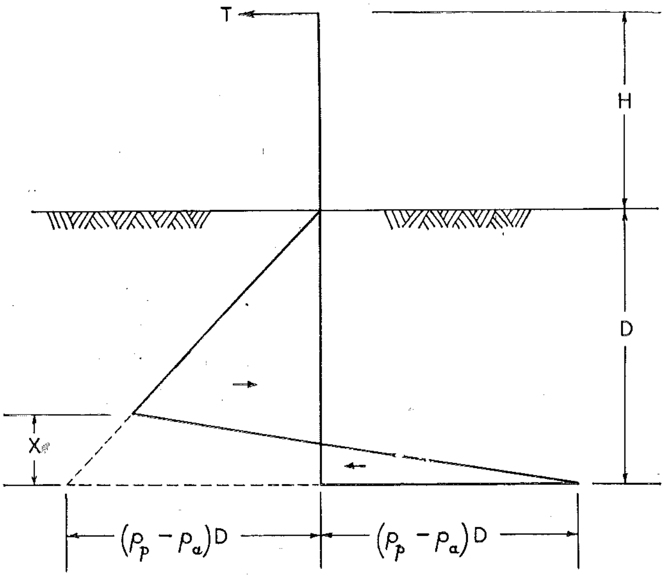


FIG. 35.

The case will now be considered of a single horizontal load applied at the top of the piling, as in *Fig.* 35.

Equating horizontal forces, $T + \dfrac{2(p_p - p_a)DX}{2} - \dfrac{(p_p - p_a)D^2}{2} = 0$. (18)

$$\therefore X = \frac{(p_p - p_a)D^2 - 2T}{2D(p_p - p_a)} \quad . \quad . \quad . \quad . \quad (19)$$

Equating moments of these forces about the foot of the sheeting,

$$T(H + D) + \frac{(p_p - p_a)DX^2}{3} - \frac{(p_p - p_a)D^3}{6} = 0 \quad . \quad . \quad (20)$$

Substituting for X,

$$HT + DT + \frac{\{(p_p - p_a)D^2 - 2T\}^2}{12D(p_p - p_a)} - \frac{(p_p - p_a)D^3}{6} = 0.$$

Extending,

$$12HTD(p_p - p_a) + 12D^2T(p_p - p_a) + D^4(p_p - p_a)^2 - 4D^2T(p_p - p_a) + 4T^2 - 2D^4(p_p - p_a)^2 = 0.$$

Simplifying and changing signs,

$$D^4(p_p - p_a)^2 - 8D^2T(p_p - p_a) - 12DHT(p_p - p_a) - 4T^2 = 0.$$

Dividing by $(p_p - p_a)^2$,

$$D^4 - \frac{8D^2T}{(p_p - p_a)} - \frac{12DHT}{(p_p - p_a)} - \frac{4T^2}{(p_p - p_a)^2} = 0 \quad . \quad . \quad (21)$$

Substituting values of p_p, p_a, H, and T gives a simple expression in D^4 from which D is obtained.

Anchored Sheet Walls.

The provision of anchorages tying back the top of the wall results in much more efficient construction, since it enables a considerable reduction in the necessary penetration to be obtained. The anchorage of the ties for sheet-piled walls and bulkheads may consist of raking piles or anchor blocks, sometimes known as "deadmen," and examples are shown in *Fig.* 36. Raking piles are used 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.* 36 tension is produced

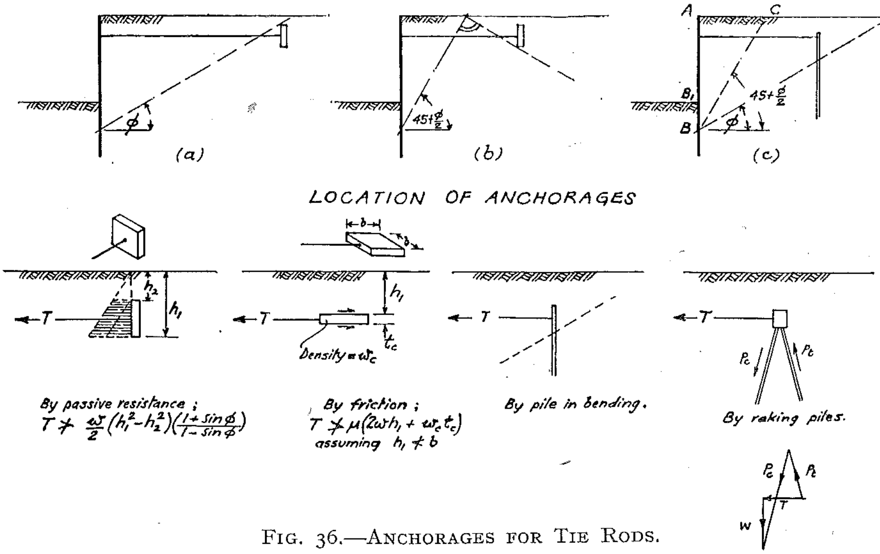


FIG. 36.—ANCHORAGES FOR TIE RODS.

in one of each pair of raking piles, so it is necessary either to have sufficient penetration of the piles to take this in friction or to add dead weight to the piles to cancel or reduce the tension.

For ordinary sheet-piled bulkheads, anchor blocks are in wider use than 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 Position of Anchorages.

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.* 36.

American and British practice have generally favoured the method shown in *Fig.* 36 (*a*), although the method indicated in *Fig.* 36 (*b*) can be used with some loss in effectiveness when space is limited. 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 enclosed by *ABC* and little in the area *CBD* the piles are then subjected to bending, and this method should generally be avoided and pairs of raking piles used instead (see also page 69). The point *B* is sometimes taken as the tip or shoe of the sheet piles, and sometimes, but incorrectly, at *B′*. Preferably *B* should be the point at which the forward movement is zero.

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.* 37, with consequent shortening and greatly

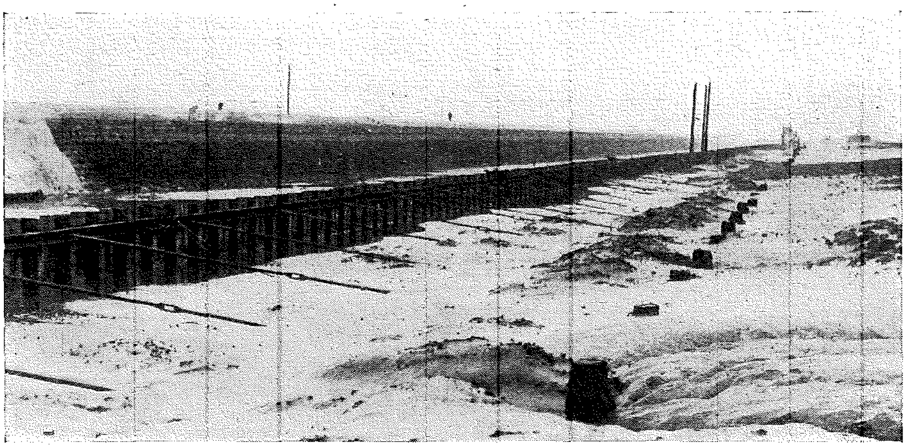


FIG. 37.—STEEL SHEET PILED RIVER WALL.

increased passive resistance of the soil at the greater depth. Since movement of the anchorage occurs as 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.

Penetration of Piling.

Anchored sheet piling 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 point of view of most efficient use of the material the results are almost identical, either providing the minimum penetration or driving to full fixity and obtaining reduced moments in the wall, in practice it is usually most convenient to extend the penetration beyond the point of simple support until the moment in the sheet wall is reduced to come just within the resisting moment of a particular section of piling. 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 overestimated, and simple support should only have been obtained instead of complete restraint, then the moments in the wall will be increased by only 40 to 50 per cent. and, as Pennoyer has pointed out, will still be within the elastic limit.

The case will be considered first where the penetration is sufficient to provide

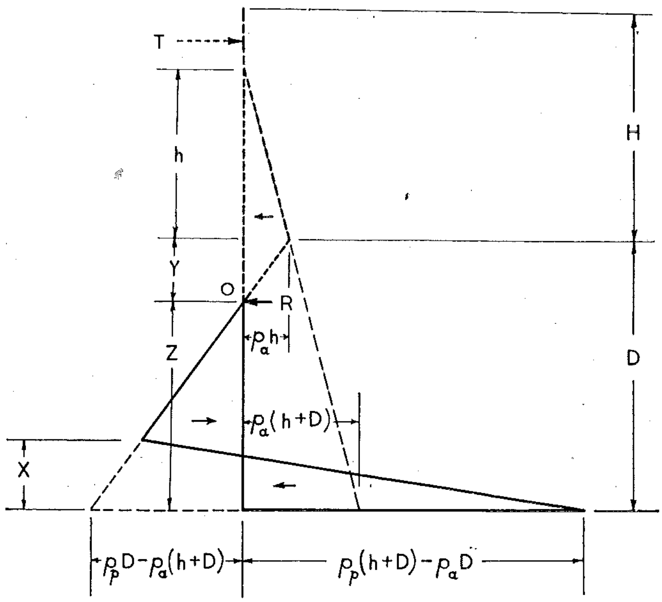


FIG. 38.

fixity at the foot of the piling. The pressures are assumed to be of the form shown in *Fig.* 38, where, as before, h is the equivalent (not the actual) height above lower ground level. It is assumed that there is a point of contraflexure at point O, the position of which is given by

$$Y = \frac{p_a h}{(p_p - p_a)} \quad \ldots\ldots \quad (22)$$

and, on this basis, that portion of the piling 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.

It will be noted that the position of point O is determined solely by the relation between the passive and active pressures above this point; while this is also a point of zero moment for the following case, in other cases where the piling is not driven to fixity the point of zero moment will be somewhat lower, approaching the foot of the piling as the degree of restraint is reduced.

Blum [15] has given the approximate location of the depth below the channel bed to the point of zero moment for uniform soil behind the wall, when driving to fixity, as follows:

ϕ (deg.)	20	25	30	35
Depth to zero moment	$0 \cdot 25h$	$0 \cdot 15h$	$0 \cdot 08h$	$0 \cdot 035h$

The procedure then is as follows.

Equating horizontal forces and multiplying by 2,

$$2R + (2D + h)(p_p - p_a)X - [(p_p - p_a)D - p_a h]Z = 0 \quad . \quad (23)$$

$$X = \frac{[(p_p - p_a)D - p_a h]Z - 2R}{(2D + h)(p_p - p_a)} \quad . \quad . \quad . \quad (24)$$

Equating moments of these forces about the foot of the sheeting, and multiplying by 6,

$$6RZ + (2D + h)(p_p - p_a)X^2 - [(p_p - p_a)D - p_a h]Z^2 = 0 \quad . \quad (25)$$

Substituting for X,

$$6RZ + \frac{\{[(p_p - p_a)D - p_a h]Z - 2R\}^2}{(2D + h)(p_p - p_a)}$$
$$- [(p_p - p_a)D - p_a h]Z^2 = 0.$$

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

$$(6RD - 6RY)(2D + h)(p_p - p_a) + [(p_p - p_a)(D^2 - DY) - p_a(Dh - hY) - 2R]^2$$
$$- [(p_p - p_a)^2 D - p_a(p_p - p_a)h](2D + H)(D^2 - 2DY + Y^2) = 0,$$

from which is obtained

$$D^4[-(p_p - p_a)]$$
$$+ D^3[(2Y - h)(p_p - p_a)]$$
$$+ D^2\left[(2hY - Y^2)(p_p - p_a) + h^2 p_a + 8R + h^2 \frac{p_a{}^2}{(p_p - p_a)}\right]$$
$$+ D\left[- hY^2(p_p - p_a) - 2h^2 Y p_a + 6Rh - 8RY - 2h^2 Y \frac{p_a{}^2}{(p_p - p_a)} + \frac{4Rhp_a}{(p_p - p_a)}\right]$$
$$- 6RhY + h^2 Y^2 p_a\left(1 + \frac{p_a}{(p_p - p_a)}\right) - \frac{4RYhp_a}{(p_p - p_a)} + \frac{4R^2}{(p_p - p_a)} = 0 \quad . \quad . \quad (26)$$

An example of such a case is given later.

For cases where a graphical method is not justified, and where the depth Y to the point O (*Fig.* 38) is known, the following formula, after Dr. Blum, may be used to obtain the penetration D necessary to obtain fixity:

$$D = K\left(Y + \sqrt{\frac{6R}{p_p - p_a}}\right) \quad . \quad . \quad . \quad . \quad . \quad (27)$$

in which K is a factor which is normally $1 \cdot 1$ but may rise to, but not exceed, $1 \cdot 2$ 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_p is Rankine's value multiplied by M (see *Fig.* 31).

When circumstances make it preferable to drive to sufficient depth to take full advantage of the restraining action of the earth, and the soil is uniform below the point O so that suitable values of p_p and p_a can be used, the portion of the wall above O can be treated as a beam with simple support at O. In other words, provided the value of R is obtained for the loading above the point O, taking account of the anchor tension and any superimposed loading, the portion of the wall below O can be treated quite separately when driven to full restraint.

Graphical Method for Anchored Walls.

The following examples of the graphical method are based upon the assumptions given earlier, using Professor Jenkin's formula for the active earth pressure and Rankine's for the passive resistance. The method here shown follows in the form of the pressure diagram the method of Dr. Blum, but with regard to penetration it is based upon the net penetration without increase for a factor of safety. The final moment diagram is subsequently reduced by the reduction factor discussed later.

The method will be apparent by reference to *Figs.* 39 and 40, except possibly the construction of the vector diagram. For this the pole distance is

$$\frac{\text{scale of moments}}{\text{scale of lateral loads} \times \text{scale of lengths}}.$$

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 has cohesive properties, but if it is not known to have any in the most serious condition of loading for the wall it is suggested that passive resistance be assumed not to commence until 1 ft. or 2 ft. below the surface of the channel bed, as a combined rough allowance against erosion and over-stressing the soil near to the surface.

CONSTRUCTION of *Fig.* 39.—Construct a unit pressure diagram down to point O. Divide 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 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 of contraflexure. This is 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.

Calculate
$$\bar{h} = \frac{\text{pressure at dredged level}}{p_a};$$
$$Y = \frac{\text{pressure at dredged level}}{(p_p - p_a)}.$$

Substitute h, Y, R, p_p, and p_a in the equation and obtain D. Calculate $p_p(h + D) - p_aD$ and $p_pD - p_a(h + D)$. Calculate X. Complete the lower part of the unit 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 zero moment at the point of contraflexure.

The scale of the moment diagram is given by pole distance $\times$ vertical space scale $\times$ scale of base of vector diagrams, as shown on *Fig.* 39.

The following method is used when it is desired to reduce the penetration to a minimum.

Since the total pressure on the front (assisted by the anchor tension) must be equal to the total pressure on the back, the back pressure at the foot is nil when the penetration is only just sufficient to prevent the wall moving forward. The problem then reduces to the following simple case.

Taking moments about T,

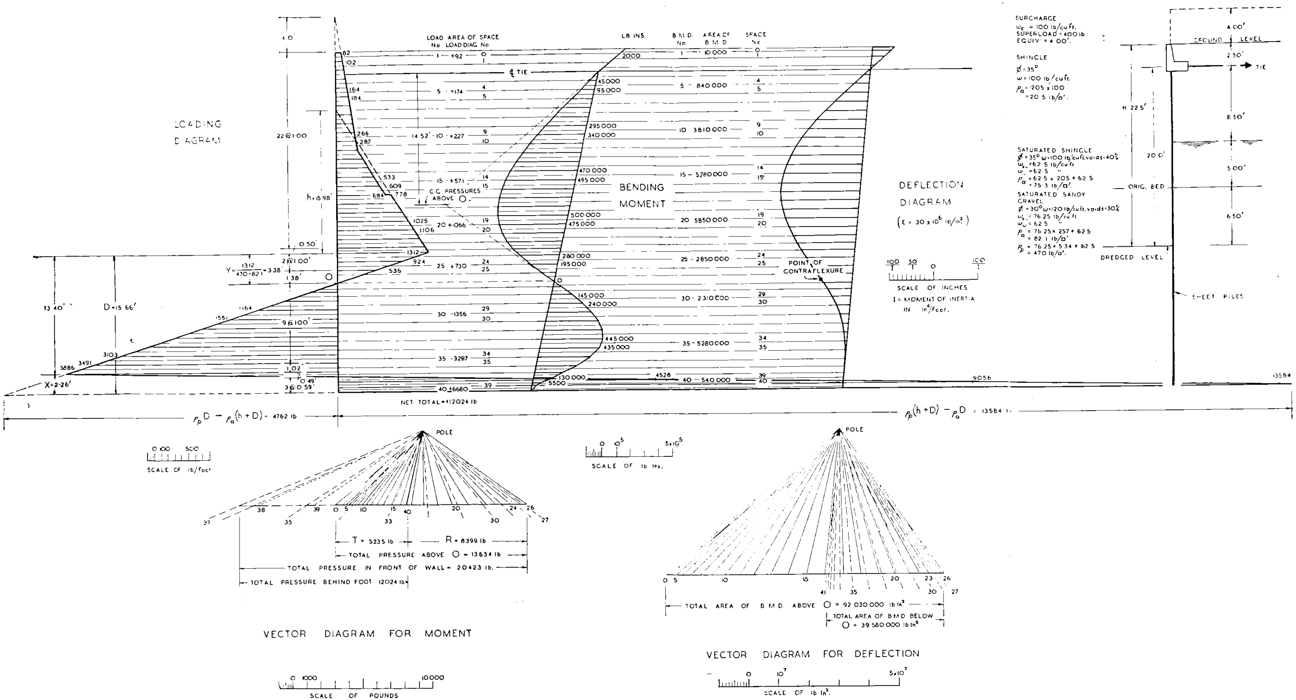
$$P_2 . g = \frac{(p_p - p_a)Z^2}{2}\left(t + Y + \frac{2Z}{3}\right) \qquad . \quad . \quad . \quad (28)$$

where P_2 is the total active pressure above point O, from which Z is directly obtained.

A graphical example of this case is given in *Fig.* 40 for the same height and loading conditions as in *Fig.* 39. 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.

Moment Reduction for Flexible Walls.

The Danish rules [16, 17] 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 the use of Jenkin's formula already recognizes this the writer would not recommend for general use the combination of a reduc-



Scale for Moment Diagram = (Pole distance × vertical space scale × scale of base of vector diagram).
Scale for Deflection Diagram = Pole distance × vertical space scale × scale of base of vector diagram) ÷ $E.I.$

FIG. 39.—ANCHORED STEEL SHEET WALL DRIVEN TO FIXITY.

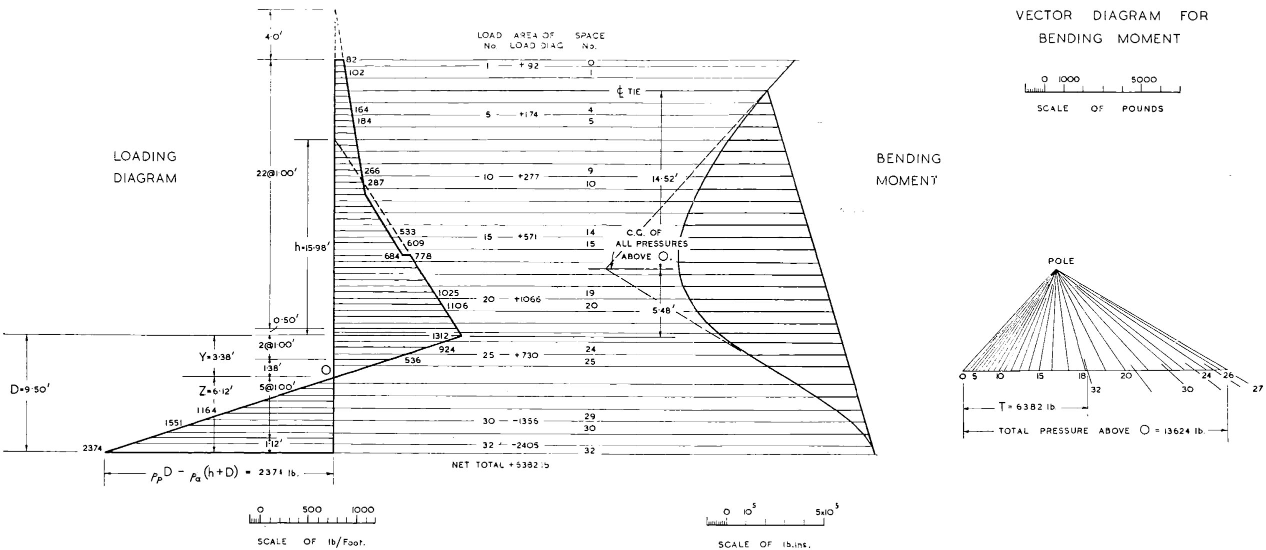


Fig. 40.—Anchored Steel Sheet Wall with Minimum Penetration.

tion of the total pressure by the Danish rules and the use of Jenkin's active pressure values.

In *Fig.* 41 the line *ADFO* represents the active pressure, assuming 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'*, with the curve *OM'A'* a parabola with horizontal axis such that

$$MM' = q = k\left\{\frac{\dfrac{10l'}{l} + 4}{\dfrac{10l'}{l} + 5}\right\}p_m \quad . \quad . \quad . \quad . \quad . \quad (29)$$

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 *A'ADFOM'A'*.

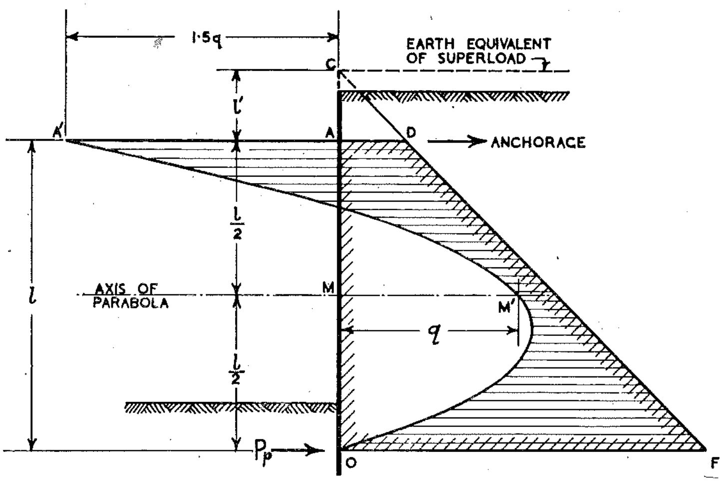


FIG. 41.—DANISH METHOD OF AMENDING PRESSURE DIAGRAM FOR FLEXIBLE WALLS.

The coefficient *k* is given by the empirical formula

$$k = \frac{1}{1 + \dfrac{0{\cdot}01}{\sin\phi}\sqrt{\dfrac{(1-n)Ea}{lf}}} \quad . \quad . \quad . \quad . \quad . \quad (30)$$

where ϕ 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 stress in the wall in bending.

For reinforced concrete values of *k* are usually between 0·7 and 0·85, while for steel sheeting the values are usually about the same or slightly higher for the same height of wall; for example, the combination of changes in *E*, *f*, and *d* often gives nearly the same value for k. Values for k for reinforced concrete walls are given in *Fig.* 42 (*a*) and for steel sheet walls in *Fig.* 42 (*b*).

The reactions in the case of simple supports are, by these rules,

$$R_A = \frac{1}{12} ql$$

$$\text{and } R_0 = \frac{1}{3} ql\,;$$

and the moment at M due to the load area $A'AOM'$ is $\frac{17}{192} ql^2$, again assuming

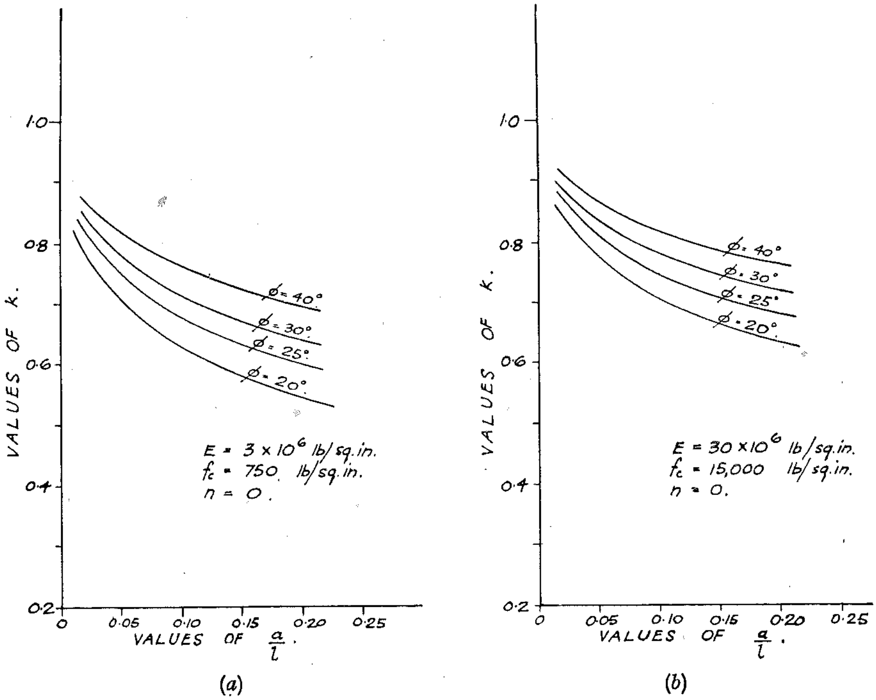


FIG. 42.—COEFFICIENT K FOR OBTAINING MOMENT REDUCTION FACTOR BY DANISH METHOD.

simple supports, which would for simplicity be often used, small negative moments at A and O being disregarded. It should be noted, however, that the gross load area $ADFO$ in this case exceeds, near the bottom, the load area by other methods.

A simplification of the method of obtaining the reduced moment and ignoring any reduction in the total horizontal force (which might be applicable to Rankine's or Coulomb's earth pressures, but not to Professor Jenkin's) can be obtained by the use of the graph *Fig.* 30 (after Mr. R. N. Stroyer,[18]) and is based on a reduction factor* of $\frac{2f}{(1 + f^{1 \cdot 5})}$ for walls of a thickness of one-fiftieth of the

* f in this expression is the liquidity factor which may be taken at Rankine's value $(1 - \sin\phi)/(1 + \sin\phi)$.

span, and reducing as the wall becomes thicker and less yielding. The form of this graph is substantiated by experience, but more recent work by Mr. Stroyer resulted in his proposing to vary the reduction factor slightly to $\frac{2f}{1+f^2}$ for the thin walls, though the writer is of the opinion that the tests on which the change was proposed justified more the continued use of the earlier expression. The existence of cohesion in the soil will probably even further reduce the moments, but no figures are available and those given can reasonably be assumed to apply.

Using *Fig.* 30, and assuming that the sheet piling section selected has a thickness-to-span ratio of about 0·05, the reduction of the moments shown in the moment diagrams of *Figs.* 39 and 40 will be about 45 per cent.; for example, the pile section would be designed for, say, 55 per cent. of the full calculated moments shown, which ignore arching action of the soil.

Comments on Calculated Passive Resistance.

The passive resistance behind the bottom of the wall has been taken in the examples in the same way as it is usually taken, but consideration of the preceding remarks on earth pressure will show that, although this assists calculations and the graphical method, this pressure is dependent on movement and therefore also on the soil modulus, which is seldom known.

For the back of the wall a method sometimes used is to take only a proportion of the passive resistance pressure, say one-fifth, as in *Fig.* 26 (*b*). If it is accepted that the assumed pressures of the usual graphical method as used here and shown in *Fig.* 26 (*a*) could be more reasonably taken as shown in *Fig.* 26 (*b*), then the depth of penetration to fixity for cohesionless soils would be greater than by the method used in the preceding examples. However, although cohesion is ignored in obtaining the passive resistance, we do not need to ignore entirely the effect it has of increasing the restraint on the piling. The effect of cohesion is to give a marked increase to the passive resistance near the surface, and the passive pressure diagram becomes like that of *Fig.* 26 (*c*) for which the penetration necessary to obtain full restraint will seldom be more, and may often be less, than that obtained by the method used in the examples given here.

Anchorages.

Anchor blocks are invariably of concrete poured 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.* 36 (*c*).

(*b*) The block must not slide forward. This may be resisted principally by either friction 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 to 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 [as in *Fig.* 36 (*b*)] two surfaces of contact, should allow for the reduced friction of wet soil.

For simplicity, the resistance of the anchorage blocks as given in *Fig.* 36 (*b*) is as if the anchorage block forms a continuous wall behind the sheet piling, as in fact in some cases may be necessary. For the usual case of a separate block to each tie-bar, however, the passive resistance of the soil is greater because the wedge of soil that resists being forced out fans out in plan. The increase of the passive resistance for separate blocks over the expressions given in *Fig.* 36(*b*) varies with the width of the block.

Anchor Blocks.

Model tests to ascertain the increased passive resistance of separate anchor blocks as compared with a continuous wall show that the increase is great for blocks with a narrow face and verify the effectiveness of anchorages of the open-grid type. An example of a model test made by the writer is shown in *Fig.* 43,

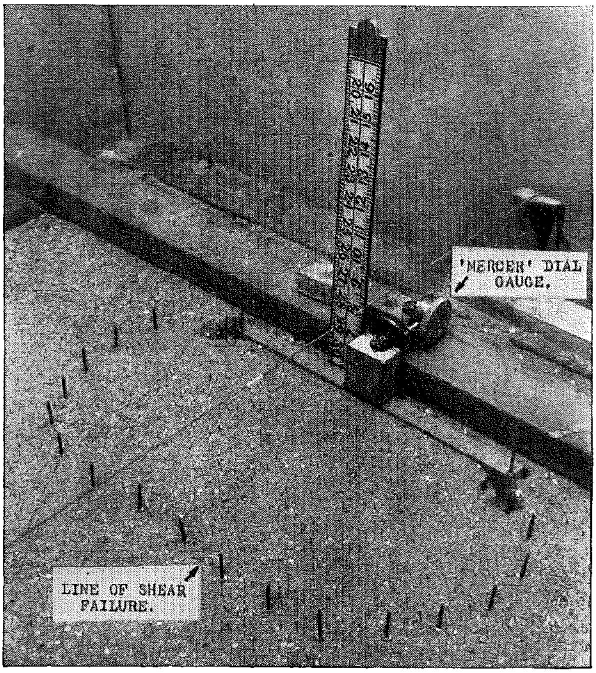


FIG. 43.—MODEL TEST SHOWING INCREASE OF PASSIVE RESISTANCE OF ISOLATED ANCHOR BLOCKS OWING TO WEDGE SHAPE OF SOIL RESISTING MOVEMENT. (NAILS SHOW LINE WHERE SHEAR FAILURE MEETS SURFACE.)

the line of the shear failure at the surface being indicated by nails, as it would not otherwise be apparent in the photograph.

From experiment, the wedge of soil forced out by anchor blocks near the surface has the shape, in section, for clean sand shown in *Fig.* 44, the angle θ

being consistently close to the angle of repose, instead of $\left(45^\circ - \frac{\phi}{2}\right)$ 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. Referring to *Fig.* 44, when the block was close to the free surface these tests showed that the

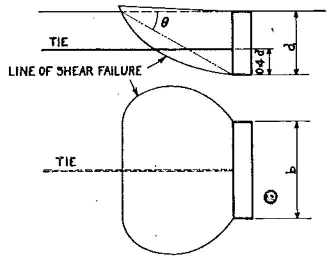


FIG. 44.—SHAPE OF WEDGE OF GRANULAR SOIL FORCED OUT BY ANCHOR BLOCK IMMEDIATELY BELOW SURFACE.

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 anchorage ; it is an unnecessary refinement except perhaps where the soil is definitely cohesionless.

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 intermediately by light vertical piles.

Alternative Graphical Method for Sheet-piled Walls.

As an alternative to the method and diagrams just described, if some assumptions are made, for example that the soil has a uniform angle of internal friction throughout the backfill and the channel bed, then the graphical method of Mr. H. D. Morgan [19] (*Fig.* 45) may be convenient. It is unusual for the soil to be uniform throughout, but if the channel bed is clean sand and the backfill is sand or a material having a slightly better angle of repose, an assumption of $\phi = 30$ deg. will not be unreasonable and with an assumed soil density of 110 lb. the results will be conservative. The further assumptions of a moment reduction factor of 0·60 and a maximum difference of water level behind the sheet piling of 3 ft. above that outside (at low tide) are both likely to be obtained with sheet piled walls of concrete or steel, while an allowance for superimposed load on backfill of 4 cwt. per square foot is generally considered ample for ordinary wharves. For these assumptions the graphical construction is then as follows :

Basis of Design :

Weight of filling . . . . 110 lb. per cubic foot

Rankine distribution used to obtain toe thrust.

Parabolic distribution used to obtain tie-rod pull.

Bending moment due to earth pressure and surcharge multiplied by factor 0·6 to allow for effect of flexible wall.

Bending Moment on Sheet Piles.—Select a tie-rod of depth d and a dredged level depth h, both measured from the cope. Set these down on a depth scale, obtaining points t and c respectively. Draw cb cutting the x-line in b : cb represents depth of point of inflexion B below dredged level.

Make ce equal to cb and draw horizontal line Be through point of inflexion, cutting bending-moment curves M_2 and $(M_1 + M_3)$ in points m_2 and m_1, m_3 respectively. Join m_2t and m_1m_3t : m_2tO and m_1m_3tO are bending-moment diagrams for sheet-piling. The horizontal intercept between m_1, m_3t or m_2t and the appropriate curve give the bending-moment values at any required depth. Curve M_2 gives the bending moment due to head of water in filling. Curve $(M_1 + M_3)$ gives the bending moment due to earth pressure and surcharge added together. The $(M_1 + M_3)$ value is subject to a reduction-factor of 0·6.

Find maximum intercepts and add 0·6 $(M_1 + M_3)$ to M_2, giving the bending moment in the sheet piling, in foot-tons per linear foot of wharf.

Tie-Rod and Pile Penetration ; Depth and Position of Anchor.—Draw a horizontal line at tie-rod depth, cutting the depth scale in p. Draw a horizontal line through B, cutting the total thrust curve and depth-scale in f and g respectively. Join f to p and make gk parallel to fp, cutting the scale of the tie-rod pull in k. Qk represents the pull in the tie-rod, in tons per linear foot of wharf.

Draw the vertical kn through k, cutting the curve of anchorage depth in n. kn represents the minimum depth of anchor required. Draw a horizontal line through n, cutting the fixed vertical LN in N.

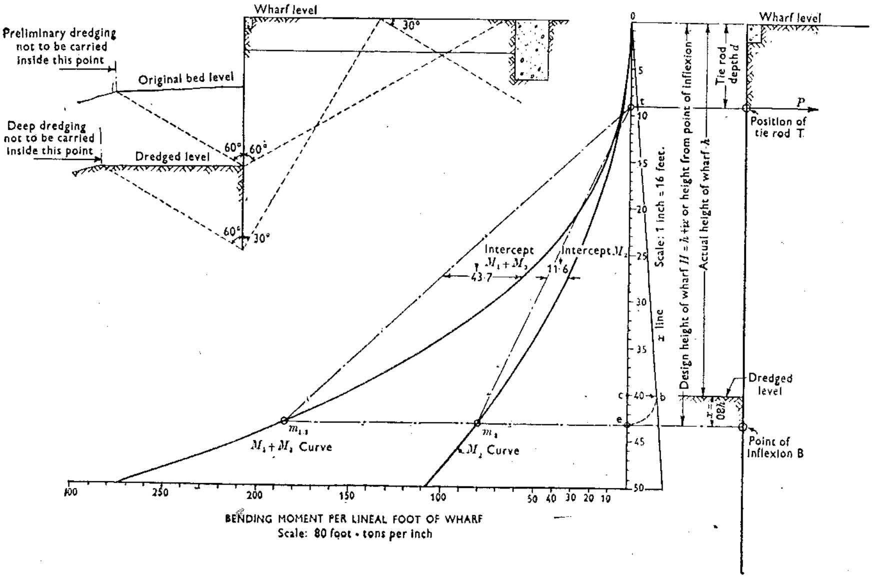
Draw the vertical from f cutting the centre of the pressure-curve in l and the thrust scale in s. Cs represents the total thrust in tons per linear foot of wharf.

Draw a horizontal line lrq cutting fp in r and the depth scale in q. Draw a vertical line through r, cutting the thrust scale in u. Cu represents the thrust at the toe of the piling in tons per linear foot of wharf.

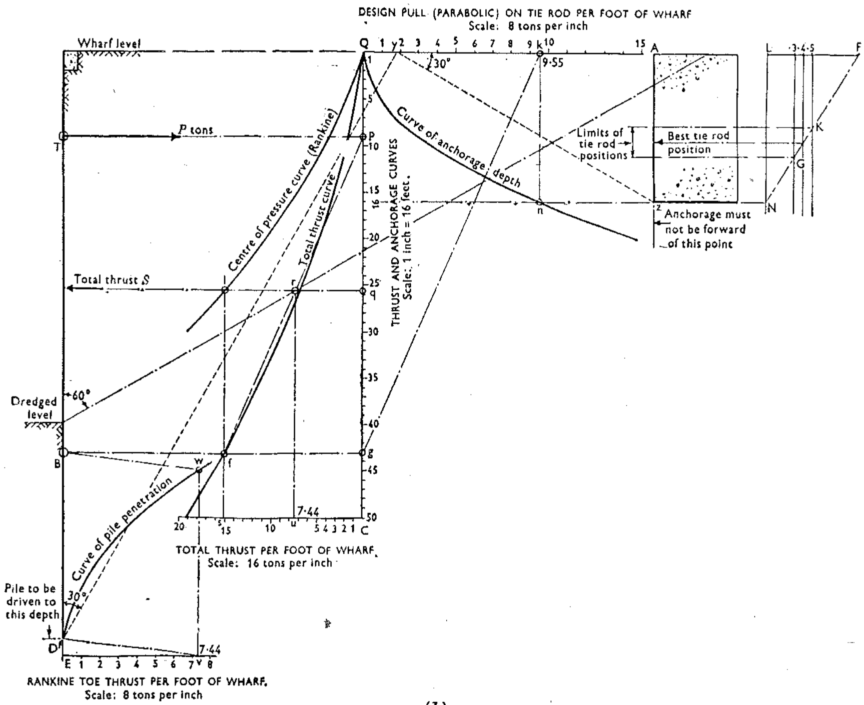
Set out Ev on the toe-thrust scale (note the scale is doubled so that Ev = twice Cu). Draw vertical vw, cutting the curve of penetration in w. Join wB and make vD parallel to wB, cutting the piling at D. D gives the minimum penetration required.

Draw Dy at 30 deg. to the vertical, cutting the cope-level in y. Draw yz at 30 deg. to the horizontal, cutting the horizontal through n in z. Az represents the face of the anchor in position and depth. Join NF, cutting the proportional verticals in G and K. Horizontals through G and K give the limits of level between which the tie-rods should be placed at the anchor end.

Fig. 45 (*a*) gives the point of inflexion of the sheet piling and also bending moments due to earth pressure and surcharge on the one hand and to the



(a)



(b)

FIG. 45.—ALTERNATIVE GRAPHICAL METHOD FOR SHEET PILED WALL ASSUMING SOIL UNIFORM AND $\phi = 30$ DEGREES.

differential water pressure on the sheet piling on the other hand. The bending-moment curves were obtained by treating the sheet piling as a cantilever and ignoring the effect of the tie-rod. By using the curves in the manner set out the effect of the tie-rod is allowed for and the bending moments are obtained, a reduction factor being applied, as described, to the bending moment induced by earth pressure.

The second diagram contains curves to determine the depth of the centre of pressure, to obtain the total thrust, and to obtain the required minimum penetration of the sheet piling below dredged level. From these the pull in the tie-rod per linear foot of wharf can be obtained, based on parabolic distribution of pressure. The total thrust in the sheet piling can also be read off. This is based on linear distribution of pressure, which gives the greater value here. When the position of the toe of the piling has been fixed the location of the anchor follows, the depth being obtained from the further curve shown.

With regard to the method of locating the anchor block as shown by *Fig.* 45(*a*) reference should be made to remarks on this subject on pages 47 and 62, since the use of the dredged level from which to set out the angle ϕ has little justification with those soils into which piles can be driven.

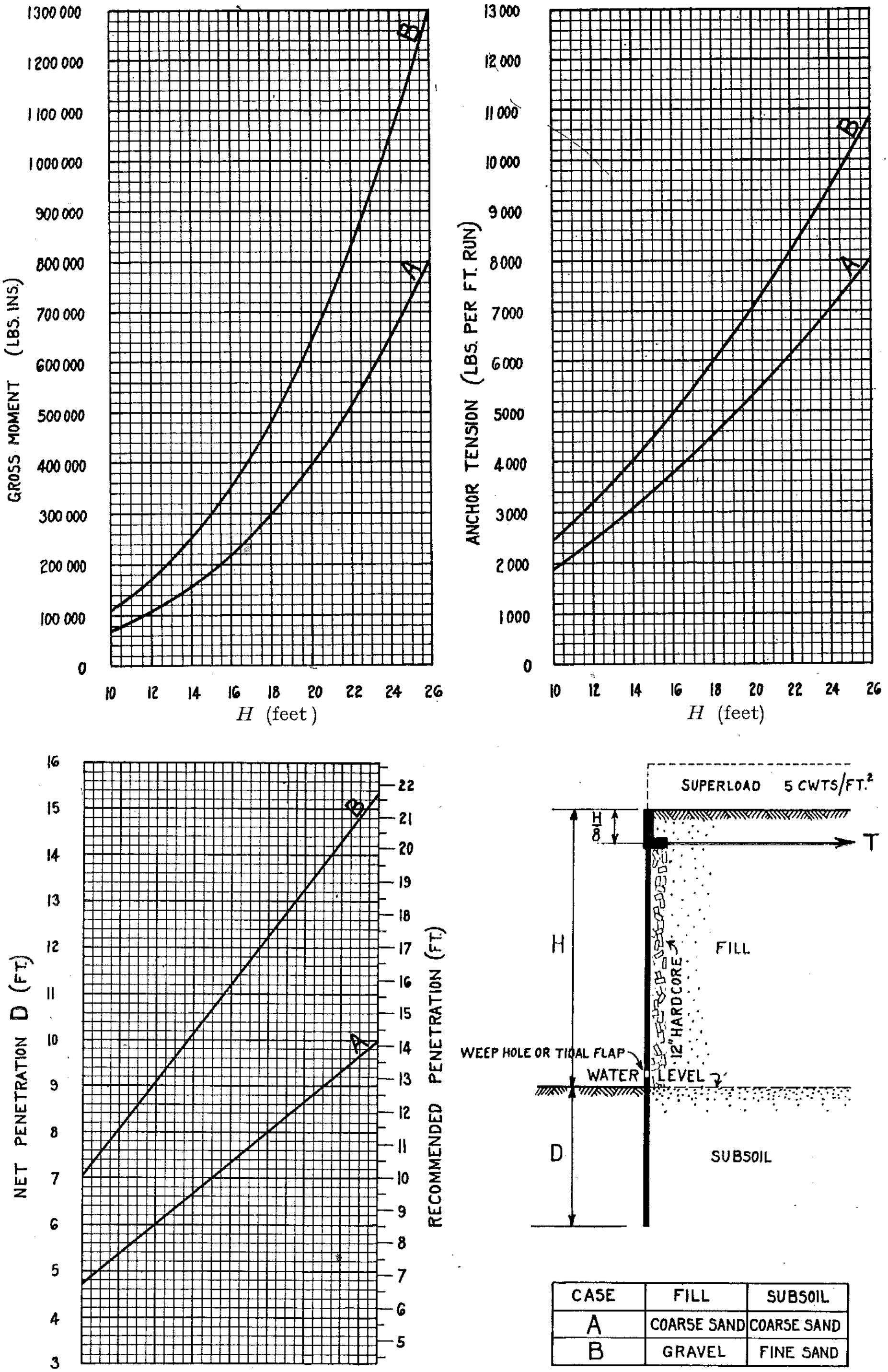
While the diagram assumes that the top of the anchor block is at the surface and extends down as far as is necessary to develop the passive resistance, the reference to the best position of the tie-rod is contingent on the anchor block being so placed. It would, however, generally be placed lower down, the tie-rod if necessary inclined accordingly, and the block reduced in size provided lowering the anchorage did not involve deep excavation in the backfill.

Design Tables.

Design tables for sheet piled 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 back-fill 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.* 46 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.

The penetration, tie-bar tension per linear foot of wall, and the gross bending moment are all calculated on the same basis as previously described, but using Rankine's values for both the active and passive soil pressure. Therefore the gross moments shown are subject to the application of the reduction factor according to the effective ratio of thickness of the wall to the effective height.

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.



CASE	FILL	SUBSOIL
A	COARSE SAND	COARSE SAND
B	GRAVEL	FINE SAND

FIG. 46.—EXAMPLES OF TIED SHEET PILED WALLS FOR TWO TYPICAL SOIL CONDITIONS, SHOWING GROSS BENDING MOMENT, TIE BAR TENSION, AND THE NET AND RECOMMENDED PENETRATION OF THE SHEET PILES ASSUMING PERFECT DRAINAGE OF THE BACKFILL.

Deep Sheet-Piled 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, 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.* 47 (*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.

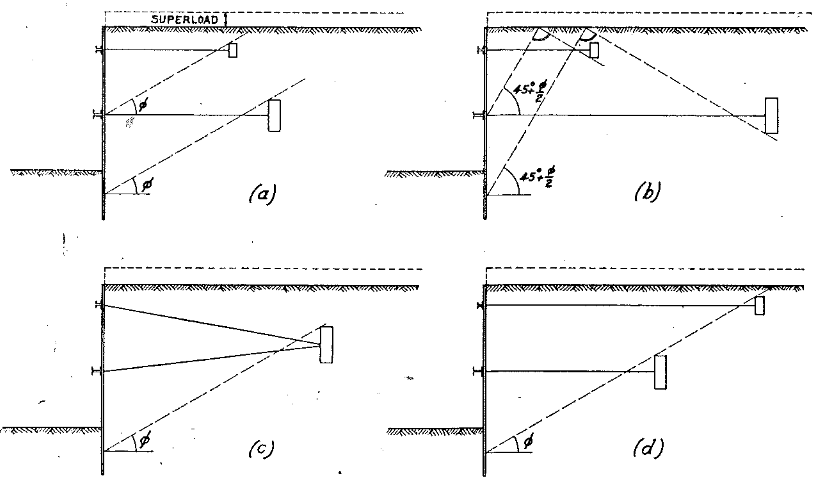


FIG. 47.

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 δ 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.* 48 (*a*)]. It should be noted that if the filling is commenced before placing the lower tie rod, causing an initial deflection δ', the force in the tie rod will be reduced in the ratio $\dfrac{\delta - \delta'}{\delta}$, since the actual deflection δ' cannot be counteracted.

(2) The reduction of tension in the top tie rod, and of pressure on the subsoil, 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 rods 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 a slightly lower value than the correct value 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 sheeting if moment reduction factors are being used. The reason for this is that the introduction of an extra tie reduces the length/thickness ratio for the piling, and the coefficients by which the gross calculated moments are multiplied will therefore often be greater than for a single tie wall.

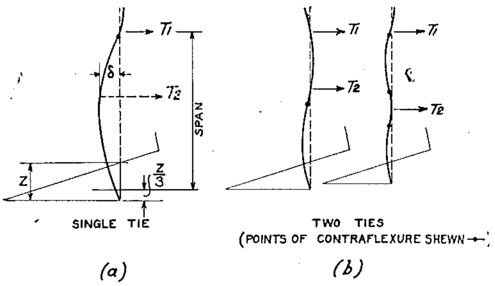


FIG. 48.

The arrangements shown at (*a*) and (*b*) in *Fig.* 47 only differ from each other in the setting out of the anchor block (see Part I). The design calculations for both are identical, but differ from those for (*c*) and (*d*). This is due to the upper anchor block being so placed that in effect the whole of the anchorage force is transferred downwards by pressure through the filling. Thus the lower tie takes the whole of the tension, and the upper tie has no effect on stability but only serves to reduce the bending moment of the upper part of the sheet piling which would otherwise act as a pure cantilever.

Compared with cases (*c*) and (*d*) the centre of gravity of the tension forces is thus much lower, 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.* 48 (*b*) that the deflected form of the sheeting, and therefore the positions of the points of contraflexure, depend on the relation 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.

The lower part of the piling is, however, quite indeterminate, as the exact pressure distribution, affected as it is by the transference of pressure from above, cannot be determined with any accuracy. It may be assumed, not unreasonably, 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.

(2) From this, obtain in the usual way the value of the anchor tension, and to this 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, 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.* 47, 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 use of a shorter upper tie than the lower is more typical of Continental practice and this method was recently 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 in showing that it is unsound to use an upper tie shorter than the lower tie. The method of *Fig.* 47 (*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.

This system of construction, of which typical cases are shown in *Fig.* 49 (*a*) and (*b*), and *Fig.* 51, takes longer to construct and is usually far more expensive than the normal anchored sheet piling, 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 avoid the likelihood of the slope flattening out after repeated changes of tide, thus increasing the pressure on the sheeting and also causing subsidence of the wharf surface behind the relieving platform.

The pressure diagrams are then as shown in *Fig.* 49 (*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.

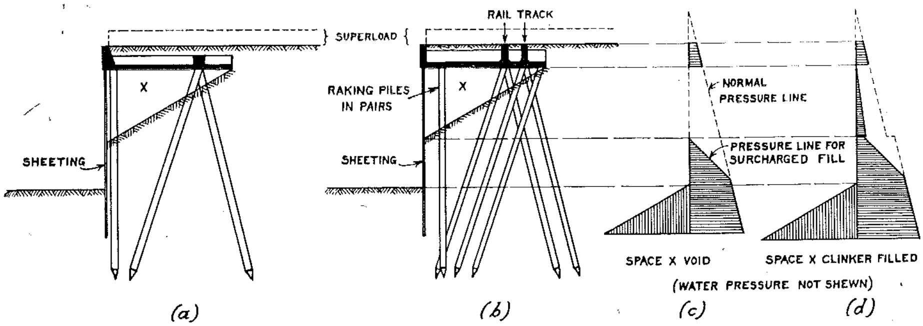


FIG. 49.

A variation of this system where the sheeting is at the rear of the platform is shown in *Fig.* 50. This has the effect of further shortening the sheeting and reducing the quantity of filling, but where it would be objectionable for the foot

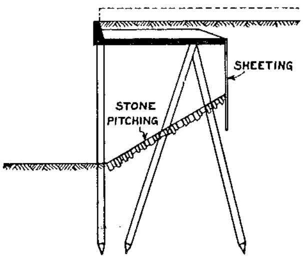


FIG. 50.

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 in tidal waters.

Sheet-piled 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 piling with earth 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-piled walls are best tied together rather than separately anchored.

Granular materials that are highly permeable are most suitable for the filling, for example coarse sand or a coarser material, but if the use of less suitable materials, such as clay, cannot be avoided, special consideration must be given to

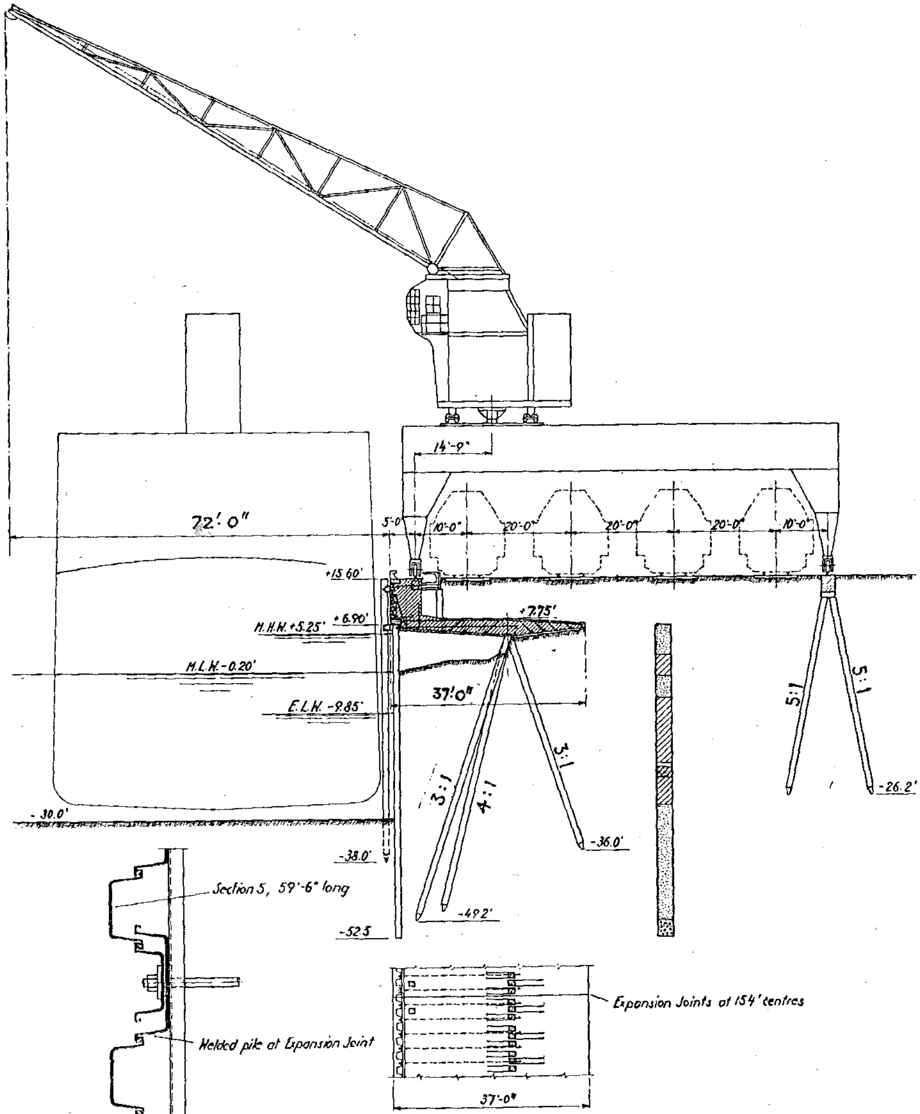


FIG. 51. [20]

the lateral pressure of the fill under the most unfavourable conditions of moisture content and frost. In this last respect the Canadian jetty [21] shown in *Fig.* 52, 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-piled walls, and the conditions for permanent stability are better than for the equivalent double-wall cofferdams, since the forces from wave pressure will usually

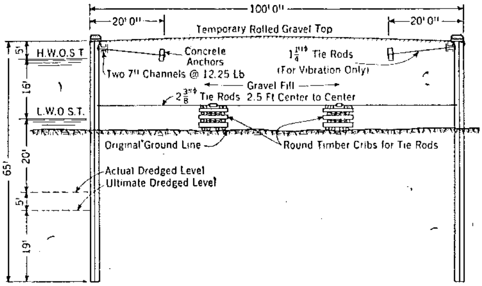


FIG. 52.—SHEET-PILED JETTY.

be negligible in comparison with those due to a substantial difference of water level.

Point Loads Behind Sheet-piled Walls.

To ascertain the stresses behind retaining walls due to concentrated loads at the surface of the backfill, methods based on assumed angles of dispersion should be considered as inapplicable, except perhaps when the soil is excessively stressed, while the application of the Boussinesq equations leads to serious under-estimation

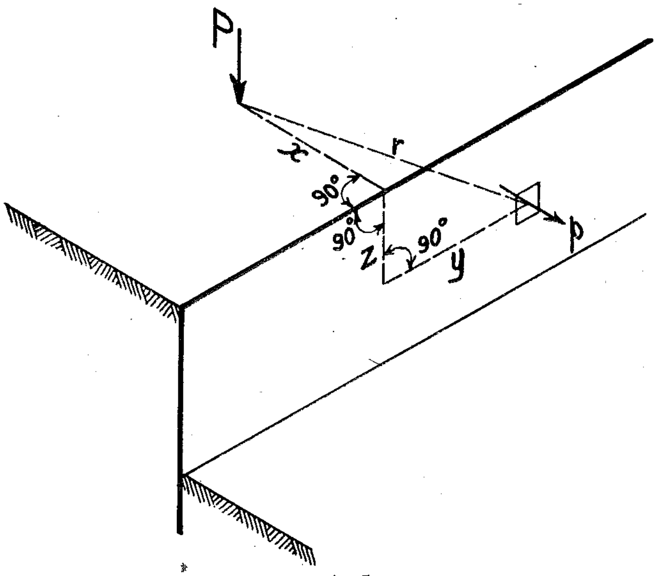


FIG. 53.—BOUSSINESQ'S CO-ORDINATES.

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 Zurich and

Spangler in America enables reasonable results to be obtained by adaptation of the Boussinesq 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 [22] for the normal unit pressure (p) on the back of the wall at a point whose co-ordinates are x, y, and z (*Fig.* 53) is

$$p = \frac{KP}{x^{0\cdot 25}} \cdot \frac{x^2 z}{r^5} \qquad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

where 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.

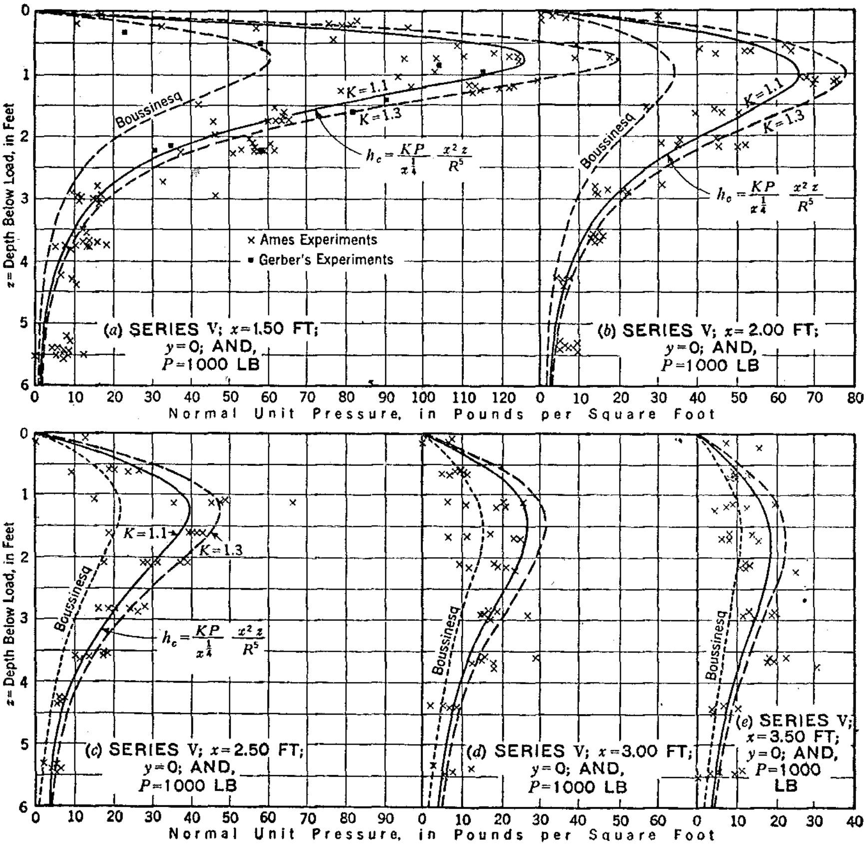


FIG. 54.—LATERAL PRESSURE ON A RETAINING WALL DUE TO A POINT LOAD ON THE SURFACE OF THE BACKFILL.

A comparison of experimental results is given in *Fig.* 54 and since, within limits, the principal of superimposition can be applied, the effect of a series of point or rolling loads can be obtained without difficulty as was shown by the writer [23] shortly after Spangler's first experiments on this subject.

In the case of crane or railway wheel loads allowance should be made for the distribution of load longitudinally by the rail and transversely by the sleepers; if details are lacking the former may be taken as 3 ft. and the latter as 2 ft. to 3 ft., both uniformly distributed. A vehicle wheel on a hard soil surface is closely similar to a point load, but distribution of the load will occur on a concrete paving as in roadway design.

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 formulæ, 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. However, in the majority of other cases, e.g. bearing piles and anchor piles, the correct estimation of the safe load is important. This subject is well covered in technical literature, and for a dynamic formula the reader cannot do better than refer to explanations of Mr. Hiley's formula [25, 26]. Mr. Hiley's formula, although not in accordance with the latest understanding of losses of energy by impact, is nevertheless the best formula available [73] and is adaptable for use with any type of driven pile. 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 per inch of lateral movement and pro rata below those 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 then to use raking piles for the anchorage.

Friction on Driven Piles.

With pairs of raking piles as in *Fig.* 36 it is necessary to determine with some care the ability of the tension pile to resist withdrawal, and since data on the resistance of piles to extraction is not so readily available in technical literature, the following may be found helpful.

A static formula used on the Continent is that of Dörr [27] which for parallel-sided piles gives an ultimate resistance to penetration (R)

$$R = wbdl \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) + \mu w l^2 (b + d)(1 + \tan^2\phi) \qquad . \quad . \quad (32)$$

where $(b + d)$ is the half perimeter of the cross section of the pile, bd is the area of cross section, l is the length of the pile in the soil, w is the density of the soil, and μ is the coefficient of friction, which for practical purposes can be taken as between $\frac{3}{4}\tan\phi$ and $\tan\phi$. For piles subject to downward axial 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. The resistance to uplift (F) will invariably exceed

$$w_1 bdl + 2(b + d)\mu\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)\frac{wl^2}{2} \quad . \quad . \quad . \quad . \quad (33)$$

It is to be noted that w_1 is the density of the material of the pile.

The resistance to extraction should always exceed this value, 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 $\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$ to $\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$, and if it is cohesive cohesion will substantially increase the apparent value of μ 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 estimations of the friction in the case of anchor piles, 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,[28] that piles driven in soft soils tend to develop a friction constant

$$f = \mu wl\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right),$$

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

$$f = \mu w\sqrt{l}\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right),$$

then the following values for f may be substituted in Mr. Porter's formula which, as adjusted by the writer, becomes

$$F = (b + d)\,fl^2 + w_1 bdl \quad . \quad . \quad . \quad . \quad . \quad (34)$$

ϕ in degrees	10	15	20	25	30	35	40
f in tons per square foot .	0·005	0·0072	0·0085	0·0095	0·011	0·015	0·020

The factor of safety the writer would suggest would be 1·5 to 2·0 for steady and varying loads respectively.

In pile-pulling tests in alluvial soil reported by Mr. Leslie Turner [29] 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 ϕ used is the angle of internal friction (see page 37) 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.

For notes on the extraction of sheet piles, see page 21.

Drainage.

Too little consideration has been given to drainage, both in text books and in practice. The best form of drainage for a sheet piled 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. 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 in the piling, but the effect of a resulting underflow on the passive resistance of the channel bed 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 adequate ; if neither, the wall will fail.

In non-tidal waters the need for drainage may not arise, and in canals it is sometimes necessary to prevent loss of water from the canal to lower land. However, normally 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 formulæ on page 78 to any particular sheet-piled 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.

Typical Failures.

Typical failures of sheet-piled walls are due to : (1) Insufficient penetration, resulting in tilting or sliding forward ; and (2) Anchorages ineffective because too near the wall, resulting first in the wall tilting and, possibly, also later in sliding forward. The form of failure in *Fig.* 27 can also be considered as insufficient penetration since if the new construction would overload a sub-soil stratum at some depth, extending the sheet piles serves to increase the radius of the cylindrical surface on which shear failure would occur. Where soft clay underlies the site of the sheet piling and the backfill becomes an additional loading, both settlement and sliding shear failure becomes possible.

Failures of sheet walls by overstressing the sheeting in bending are rare, and when they occur are probably more due to inadequate drainage than too optimistic estimation of the normal properties of the soil.

Omission to take into account arching action in the soil, and the consequent reduction in moments in the sheeting and the reduction in 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 piling for bending stresses.

PART III

COFFERDAMS

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-piled 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-piled 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.

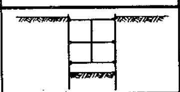
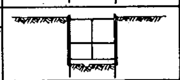
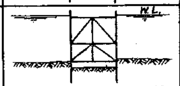
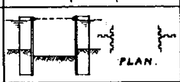
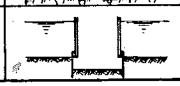
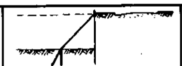
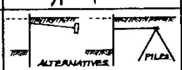
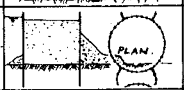
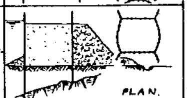
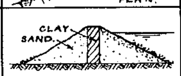
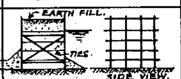
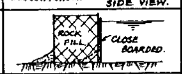
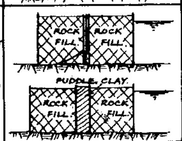
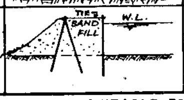
Subsequent examples will show that too many site factors are involved to make it possible to define simply the limits for the relative usefulness of the cofferdam and the various alternatives. Generally speaking, however, the cofferdam method is more likely to be the more economical the greater the plan 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.* 55, 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 will be 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.

	LOCATION	TYPE.		SUITABLE FOR :	NOT SUITABLE FOR :	REFERENCE :
	SINGLE WALL TYPES. (FOR INDIVIDUAL PIERS OR FOUNDATIONS).					
kuip	ON LAND.	STEEL SHEET PILE.		AVERAGE CONDITIONS, AND ALSO FOR DEEP EXCAVATIONS IF SHEETING PENETRATES INTO CLAY OR EQUAL IMPERMEABLE STRATA.	SUBSOIL WITH BOULDERS BELOW SUBSOIL WATER LEVEL. DEEP EXCAVATIONS IN FINE SAND UNLESS LOW HEAD OR SIMULTANEOUS GENERAL LOWERING OF W.L. BY PUMPING. EXCEPTION GOOD SEAL OF SHEETS IN CLAY.	1
"		STEEL OR TIMBER PILES & HORIZONTAL SHEETING.		DITTO.	DITTO.	2
"	IN WATER.	SHEET PILE.		AVERAGE CONDITIONS, ESPECIALLY IF SHEETS SEALED IN CLAY.	DEEP WATER UNLESS PILES INTO CLAY	3
		SHEET PILE WITH BUTTRESSING.		STILL & SLOW MOVING WATER WHERE CLEAR INSIDE WORKING SPACE ESSENTIAL.	EXPOSED HEIGHTS SUFFICIENT TO OVER STRAIN THE INTERLOCKS, ALSO POOR SOIL IN CHANNEL.	4 *
		CRIB.		ROCK BOTTOM & RISK OF FLOODING.	MOST OTHER SOILS.	5
kuip (Beaudemoulin)		MOVABLE.		SEVERAL USES ON SAME SITE.	WATERWAYS WITH STRONG CURRENTS.	6
	TYPES FOR DE-WATERING LARGE AREAS					
kuip geschoord	ON LAND.	SHEET PILING BUTTRESSED.		SOFT BOTTOM AND WHERE TOP SURFACE NOT ACCESSIBLE FOR ANCHORING.		7
" verankerd		SHEET PILING ANCHORED.		NORMAL CONDITIONS.		8(a) & 8(b)
		SHEET PILING BUTTRESSED.		ROCK BOTTOM.		9
cellendam kistdam	SLOW FLOWING & STILL WATERS.	CELLULAR.		ROCK BOTTOM	WHERE SPACE DOES NOT PERMIT BERM SUFFICIENT TO REDUCE HYDRAULIC GRADIENT OF SEEPAGE WATER.	10 *
"		CELLULAR.		ROCK BOTTOM	WHERE DRIVING IS HARD & INTERLOCKS MAY BE TORN.	11 *
gronddam	STILL & FLOWING WATER.	EARTH.		SHALLOW OR TIDAL WATERS.	SOFT CLAY SUB-STRATUM.	12
		OHIO TYPE. (NAVIGABLE WATERWAYS)		ROCK BOTTOM. (CONSTRUCTION FROM BARGES).	SOFT BOTTOM UNLESS AMPLE PROTECTION AGAINST EROSION.	13
		CRIB.		ROCK BOTTOM AND SWIFT CURRENT.		14
		INTERMITTENT DOUBLE CRIB. (& STAGGERED CRIBS).		ROCK BOTTOM. SWIFT CURRENT AND POSSIBILITY OF FLOODING.		15(a) 15(b)
		SHEET PILING.		NORMAL CASES OF SOFT BOTTOM	PENETRATION OF PILES LIMITED BY ROCK BOTTOM CLOSE TO CHANNEL BED	16

* THESE TYPES ONLY SUITABLE FOR USE WITH PILE SECTIONS HAVING STRONG INTERLOCKS.

FIG. 55.—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 so that the structure will have equal earth pressures on the opposite sides.

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.* 55) 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.

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 in Part I.

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 piling is 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 worthwhile 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.* 55) is a double line of sheet piling with soil filling (*Fig.* 56), 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.* 56, 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 its effect, previously mentioned, of reducing the angle of internal friction. This case is virtually the same as two anchored sheet pile walls back to back, already dealt with in Part II, except that moment reduction factors cannot safely be applied.

Assuming that the sheet piling is driven to obtain simple support, that is, to obtain a passive resistance of the channel bed equal to the outward horizontal earth pressure, but not sufficient to provide restraint moment, then, neglecting arching of the soil between the two walls, the tension in the tie-bar will be given on the safe side by $T = \frac{p_a l^2}{6}$, where l is the effective span of the sheeting. Both arching and restraint of the piles in the channel bed would reduce the value of this tension. Some degree of arching of the soil may take place, but it is better to discount this in the case of cofferdams.

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.* 56 (*b*), and a single

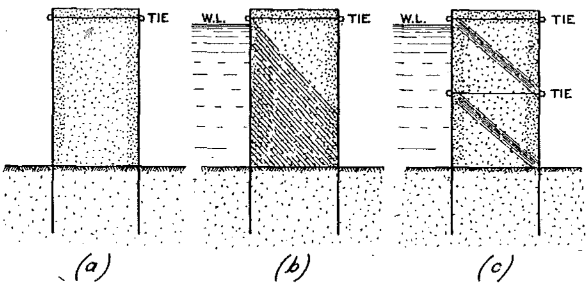


FIG. 56.—DOUBLE SHEET-PILED WALL COFFERDAM.

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.* 56 (*c*), resulting in economy of sections and increased 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 elsewhere a berm on the inside may become essential to obtain proper stability.

Watertightness.

Water will enter the cofferdam in two ways: (*a*) by leakage through the sheet piling, and (*b*) by underflow as indicated in *Fig.* 57. 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. Any small percolation through interlocks is sealed by causing fine

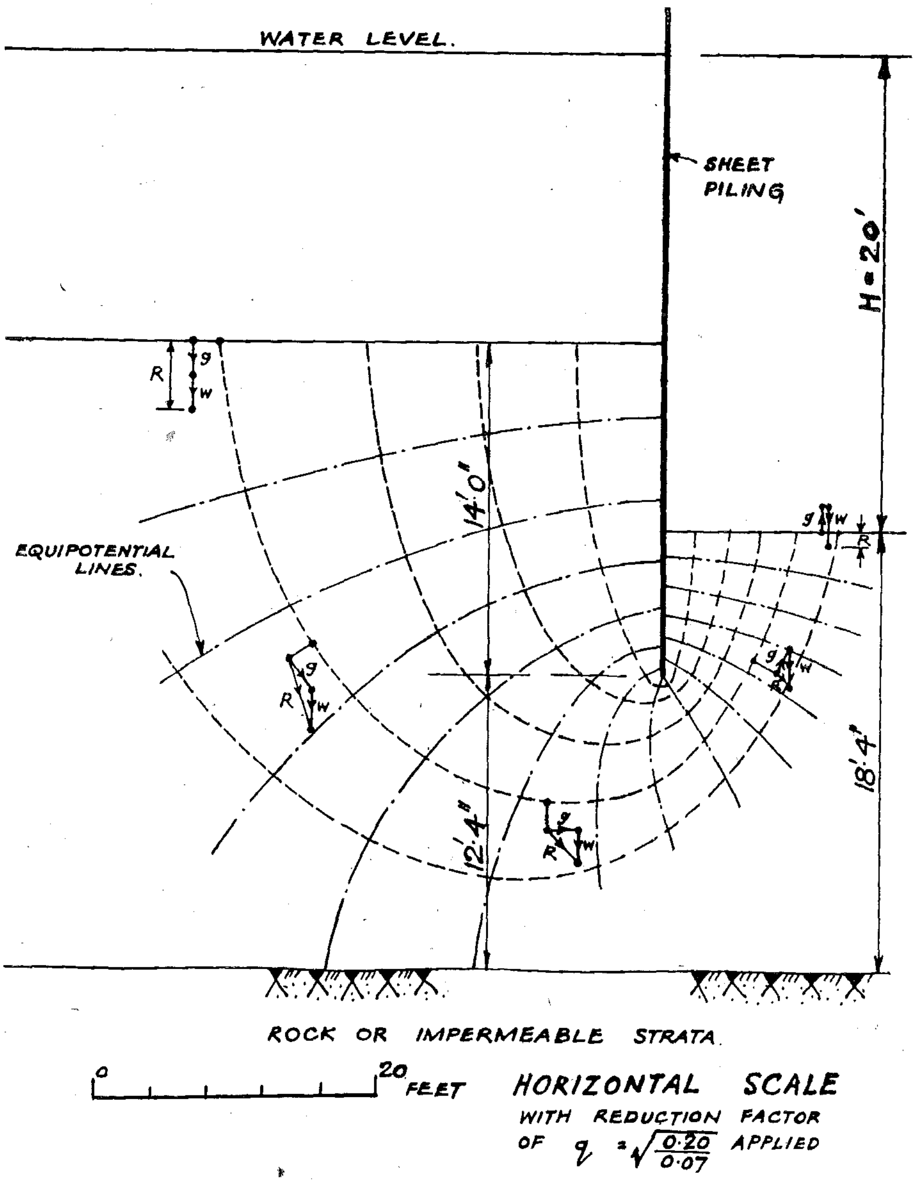


FIG. 57.—SEEPAGE BY UNDERFLOW.

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 stick and seal the leakage. 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 the 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 de-watered. This can sometimes be only suspected before the excavation has proceeded to some depth, and in such cases a " blister " 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. If the soil is permeable and the penetration insufficient, " boils " will occur in the floor of the interior, and besides the floor becoming " quick " the inflow may make de-watering impossible.

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 [30] and has been dealt with at length more recently by Harza,[31] Casagrande, and others.

A typical illustration of seepage is shown in *Fig.* 57 and follows D'Arcy's formula

$$v = KS \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

where v is the average velocity of flow, K is the coefficient of permeability,* 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.

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.* 57, 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

* See *Table* VII.

tubes, and thus the velocity increases as the flow lines get closer together, as around the bottom of the sheeting. The spacing of the flow lines is related to the spacing of the equipotential lines by $\frac{\delta s}{\delta n} = 1$, 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 impervinos 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 δH is the head lost in each curvilinear section $N\delta H = H$ and the hydraulic gradient in any section is $S = \frac{\delta H}{\delta n} = \frac{H}{N\delta n}$, and 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:

$$Q = \frac{m}{N} KH \qquad (36)$$

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. Samsiœ 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,

$$K' = \sqrt{K_v K_h} \qquad (37)$$

and it is then necessary, in addition, to reduce the horizontal scale by a distortion factor which, assuming K_h is, as is usual, greater than K_v, is

$$q = \sqrt{\frac{K_h}{K_v}} \qquad (38)$$

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

In *Fig.* 57 (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, and w_{ew} 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 $S = 1{\cdot}0$ the sand grains float and boils develop.

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_c is the flotation gradient, p the porosity of the soil, and G the specific gravity,

$$F_c = (1 - p)\ (G - 1) \qquad (39)$$

The values from Lane [32] and Wing in *Table* VII 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 VII.—COEFFICIENTS OF PERMEABILITY, K, FOR TYPICAL SOILS.

Soil	Cubic cm. per second per cm.² for 1 cm. thickness and 1 cm. head*
River sand	0·041 to 0·266
Dune sand	0·0185
Beach sand	0·0089 to 0·0216
Undisturbed soil, fine sand to gravel .	0·0163 to 0·316
Clay	0·000000023 to 0·000023

* 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 12·3.

EXAMPLE.—Assume that the sheeting is an impervious continuous straight barrier with dimensions as shown in *Fig.* 57, and that, due to stratification, $K_v = 0{\cdot}07$ and $K_h = 0{\cdot}20$. Then $K' = \sqrt{0{\cdot}07 \times 0{\cdot}20} = 0{\cdot}116$ cu. cm. per second per square centimetre, or $0{\cdot}116 = 12{\cdot}3 = 1{\cdot}43$ gall. per minute per square foot. Now from *Fig.* 57, $\frac{m}{N} = \frac{6}{14} = 0{\cdot}428$, and from equation (36) $Q = 1{\cdot}43 \times 0{\cdot}428 \times 20$ $= 12{\cdot}2$ 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.* 91 and 92), 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 construction of a diagram as *Fig.* 57 is necessary in all but simple cases of the type shown.

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 1,000), and θ is the temperature in degrees Fahrenheit, then with the effective grain size D_{10} in millimetres,

$$V' = \left[C(D_{10})^2\right] \frac{(\theta + 10)H}{60L} \qquad . \quad . \quad . \quad . \quad (40)$$

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 representative sample of the soil, e.g. the value of D_{10} is 0·2 for the sample of sand at Trempeleau, Wisconsin, shown in *Fig.* 58.

The flow of water into a cofferdam can be estimated by using Hazen's formula (40) in combination with the flow net diagram, each stream flow band being dealt with separately, and L in the formula being the total length of the seepage path in each case. 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.

Messrs. White and Prentis,[33] 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 measure-

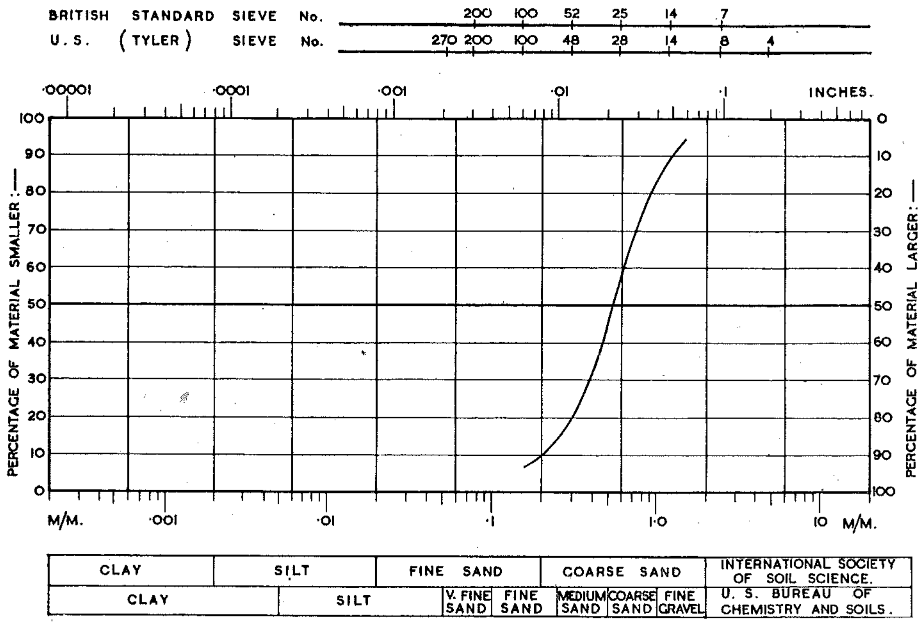


FIG. 58.

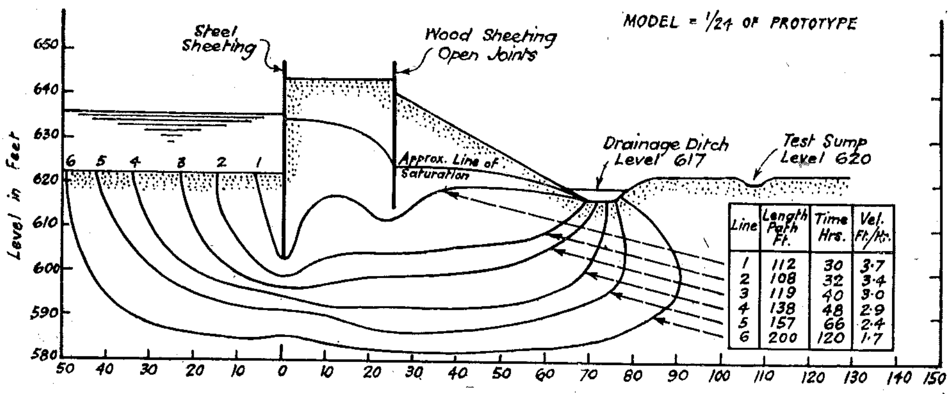


FIG. 59.—FLOW NET AS DETERMINED BY MODEL STUDY.

ments of the flow of water under the cofferdam at Trempeleau, Wisconsin, including the effects of varying the flow by increased penetration of the sheet piling. The result of these measurements is shown in *Fig.* 59, the grain size distribution curve for the sand being that given in *Fig.* 58.

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 recently at Plymouth.[34] If the position of the resulting " blows " can be fairly closely determined on de-watering 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 to be 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 de-watered from points not too close to the sheeting.

The height of a cofferdam will be determined with a view to preventing the

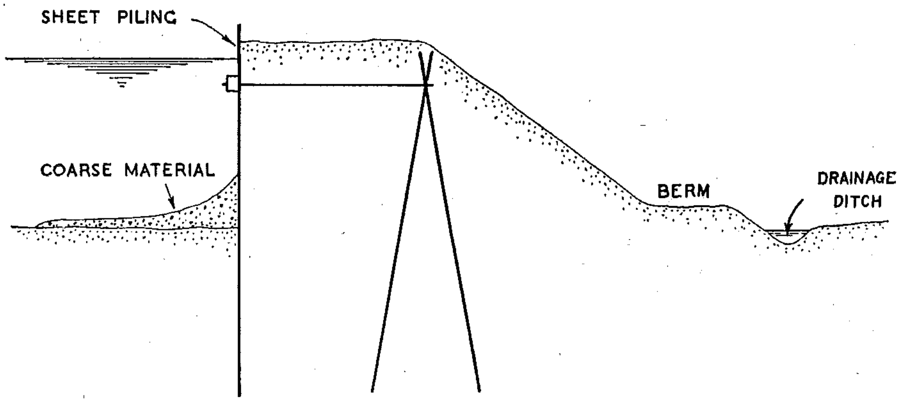


FIG. 60.—USE OF BERM TO FLATTEN HYDRAULIC GRADIENT OF SEEPAGE WATER.

inflow of water by over-topping during floods. 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 the diagram (*Fig.* 60). An adequate berm to reduce 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 wellpoint 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 lead 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 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 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 have a great effect on the wear of 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 3,000 to 6,000 gallons per hour respectively for 3-in. and 4-in. suction pipes.

Wellpoints.—An advantage of the wellpoint system 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 wellpoint equipment is shown in *Fig.* 61, the wellpoint and riser pipes being jetted down to the desired level. The jetting head contains a rubber ball acting as a non-return valve, which is kept off its seat during sinking but which closes by suction during subsequent pumping, so that all incoming water passes through fine gauze screens before entering the $1\frac{1}{2}$-in. internal diameter riser pipes. At the tops of the riser pipes swing connectors are provided, connecting with a header pipe serving all the risers. The pumping unit consists of a large-capacity vacuum pump driven by an internal combustion

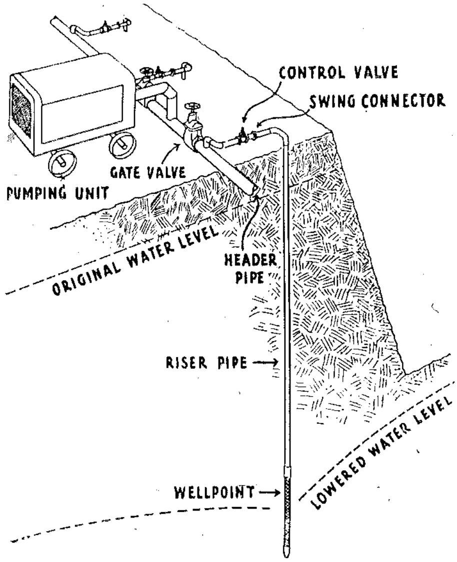


FIG. 61.—DIAGRAM OF THE WELLPOINT METHOD OF PUMPING.

engine, while for jetting down the wellpoints a separate centrifugal water pump set is used capable of delivering up to 300 gallons of water per minute at 150 lb. pressure.

Using this method it is possible to have a great number of riser pipes surrounding the area of a land cofferdam, thereby reducing the water table considerably before commencing excavation. The wellpoint system is to be preferred to pumping from the inside of a land cofferdam if there is then risk of the fine material escaping in the water, since by the wellpoint method the water pumped should be quite clear after the first few minutes. The spacing apart of the wells (shown diagrammatically rather close in *Fig.* 61) may, in practice, become about 40 ft. apart along the edges of the proposed excavation.

By a development of this method used to avoid, with deep pumping, the lowering of the sub-soil water in depths suitable for a suction pump at the surface, say in lifts not exceeding 18 ft., a well of 20 in. or even larger diameter 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 pump may then be lowered in each of the 14-in. diameter pipes. The first 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.

A formula quoted by Sir Henry Japp[24] 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.* 62)

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.

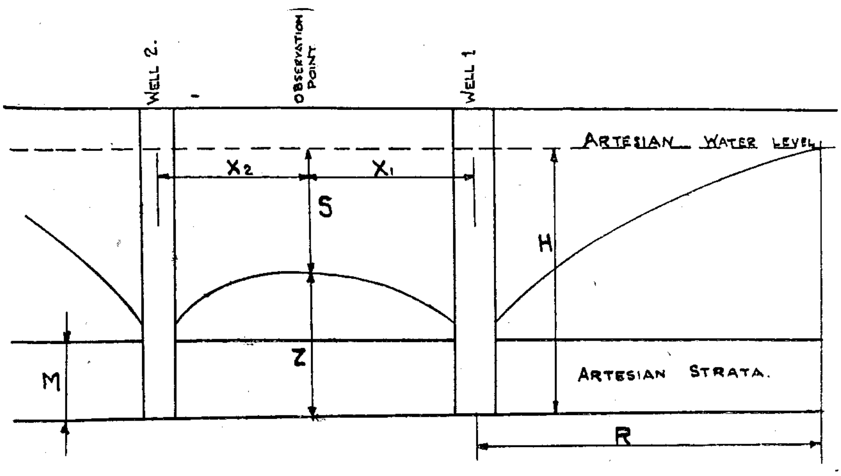


FIG. 62.—WELLPOINT METHOD OF PUMPING.

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. This method can also be expected to be the cheapest in the majority of cases. However, the wellpoint system is gaining in popularity, and in difficult work the advantages will be apparent.

EFFECTS OF PUMPING.—With land cofferdams some thought has to 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.* 63) 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

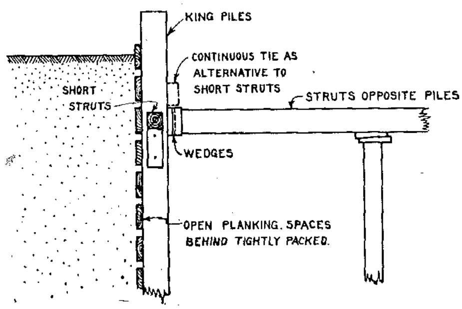


FIG. 63.—OPEN SHEETING METHOD OF EXCAVATION.

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."

Effect of Vibration on Existing Structures.

The driving of sheet 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, there are great 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 to neighbouring structures.

With regard to the direct effect of the driving of sheet piles, in the case of steel sheeting there is seldom any trouble, but if piles of other materials are driven into clay there is the possibility that the soil is 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 any drains and other pipes nearby.

Cofferdams in Flowing Waterways.

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

A danger that may arise when the cofferdam is over-topped, say during a flood, is through the washing-out effect of the great volume of water which may pour over the top of the sheeting. With an earth fill 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 up-stream

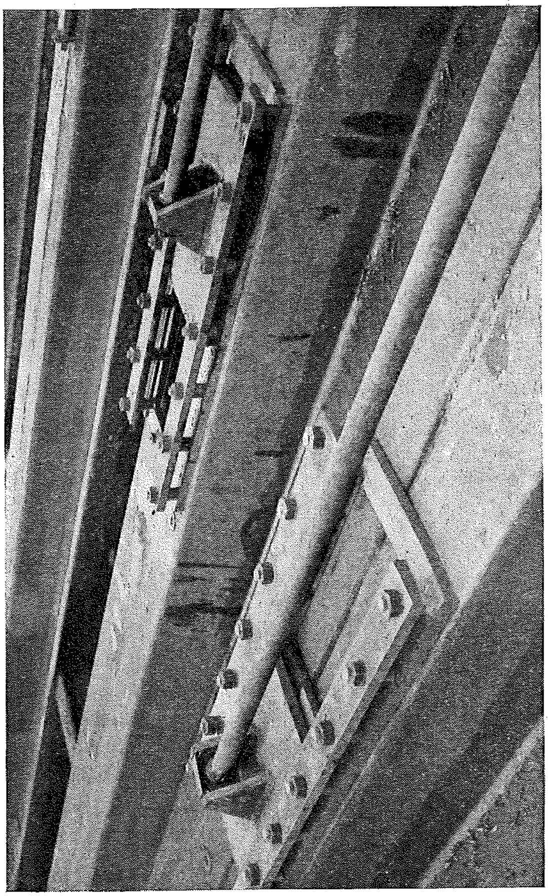


Fig. 64.—Sluice Gates in Steel Sheet Piles.

side. This may wash out the berm, exposing the sheeting to greatly increased moments. 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, and 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 the water comes over the top; withdrawing one or two sheet piles is an unsatisfactory method and may be most dangerous owing to the resulting scour. Sluice gates in the steel sheeting as in *Fig.* 64, 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

$$T = \frac{0{\cdot}41A\sqrt{(H_1 - H_2)}}{A_s}$$

where A is the area of the impounded water, 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 piled 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 piled 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-piled 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 text books, 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 to be self-supporting, the latter method being adopted where there is a soft bottom or to leave the bottom clear for excavation.

Manufacturers of steel sheet piling 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,

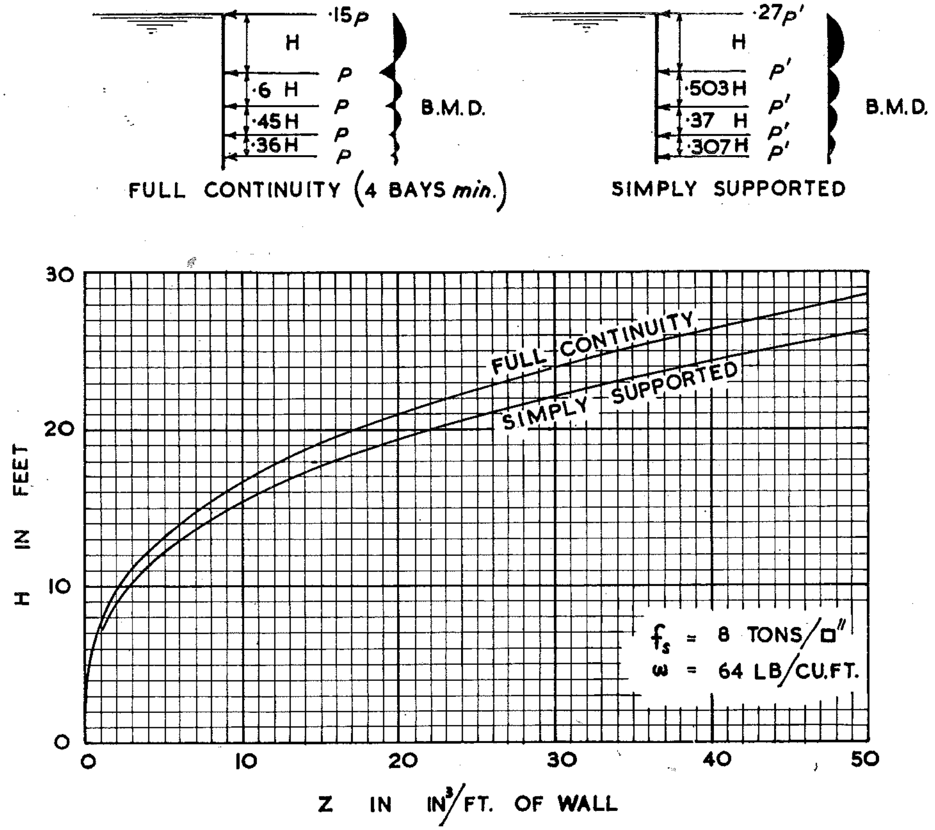


Fig. 65.—Calculated Maximum Spacing of Walings Vertically below High-water Level, Based on the Strength of the Sheet Piling.

to the reactions as calculated by the approximate method also being slightly inaccurate. *Fig.* 65 shows the maximum spacing of walings taking continuity into account, but ignoring the greater stiffness of the top support when of the same section as those below but receiving only a fraction of the lateral load of the others. Owing to the large section required for the struts when the walings are spaced at the maximum to suit the steel sheeting, if timber bracing is used the spacing of the walings vertically is usually reduced, and *Fig.* 65 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 in the steel sheeting and greater forces in the walings and struts at the level immediately above the strutting being fixed. The increase in stresses in walings and struts caused by the fixing of successive levels of bracing as the water level is lowered is of more importance the greater the depth of water, since the difference in outside and inside water level causes a pressure difference through the whole exposed height as shown at (*b*) in *Fig.* 66. For this reason, unless the method

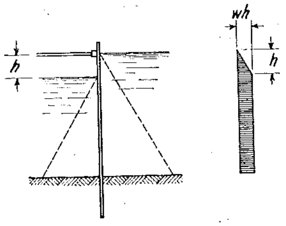


FIG. 66.—UNBALANCED HYDROSTATIC PRESSURE ON SHEET PILING AS THE WATER LEVEL IS LOWERED AND THE BRACING IS BEING FIXED IN SUCCESSIVE STAGES.

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. An alternative is 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 for 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.

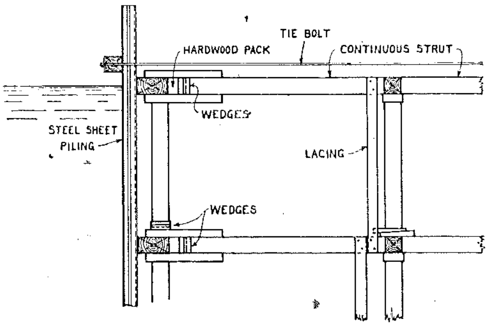


FIG. 67.—COFFERDAM BRACING WITH HARDWOOD WEDGES.

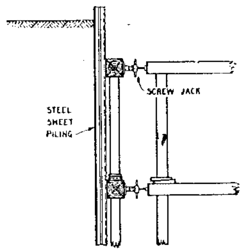


FIG. 68.—COFFERDAM BRACING WITH SCREW JACKS.

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.* 67) or metal plates with screw adjustment (*Fig.* 68) 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 so much as to 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 de-watered, of which a typical detail is given in *Fig.* 67, normally involves this temporary increase of stresses.

The method of framing and lowering the bracing before the dam is de-watered avoids consecutive increased forces in the walings and braces when the frames

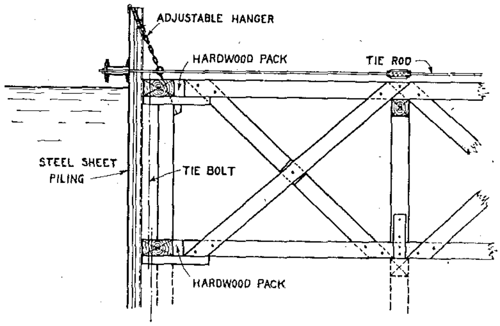


FIG. 69.—FRAMED BRACING SET IN PLACE BEFORE THE DAM IS DE-WATERED.

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.* 69. 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.* 67, 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 Specification No. 940, which closely follows the corresponding United States standard A.S.T.M. No. D245/37 and the Canadian (C.E.S.A.) standard No. A43.

An extract from Part 2 of the B.S.S. (*Table* VIII) gives the permissible stresses for beams and struts under fully-protected conditions.

TABLE VIII.

WORKING STRESSES ON TIMBER FOR CONTINUOUSLY DRY CONDITIONS.

		Beams				Struts		
		Grading schedule for large beams	Safe bending stress (lb. per sq. in.)	Shear parallel to grain (lb. per sq. in.)	Compression perpendicular to grain (lb. per sq. in.)	Grading schedule for struts	Safe compressive stress (lb. per sq. in.)	Modulus of Elasticity (lb. per sq. in.)
Douglas fir (coast)	Dense	*L**	1750	105	380	*S**	1300	1,600,000
	Close-grained	*L*†	1600	100	350	—	—	
	Dense	*M**	1400	84	380	*T**	1040	
	—	*M*	1200	80	350	*S*	1200	
Pitch pine	Dense longleaf or shortleaf	*L**	1750	105	380	*S**	1300	1,600,000
	Do.	*P**	1600	100	350	*S*1*	1200	
	Dense longleaf	*M**	1400		380	*U**	1000	
	Dense shortleaf	*N**	1200			*V**	900	
Western larch and Western hemlock		*L*	1300	85–90	300–325	*S*	1000	1,400,000
		M	1000	68–72		*T*	800	
Red pine and Ponderosa pine		*L*	1200	85	300	*S*	900	1,300,000
		M	960	68		*T*	720	
Canadian spruce, Sitka spruce and Engelmann spruce		*L*	1100	85	250	*S*	800	1,200,000
		M	800	68		*T*	640	
Eastern hemlock, Jack pine, and Lodgepole pine		*L*	1100	75–80	275–300	*S*	800	1,100,000
		M	880	60–64		*T*	640	
Amabilis fir and Balsam fir		*L*	1100	75	175	*S*	800	1,100,000
		M	800–880	60		*T*	640	

* The requirement for density in Clause 20 of B.S. 940, Part 2, 1942, also applies.
† The requirement for close-grain in Clause 21 of B.S. 940, Part 2, 1942, also applies.

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 cofferdam bracing the writer would recommend 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 waney edges, and the proportion of summer wood to spring wood. *Fig.* 70 is

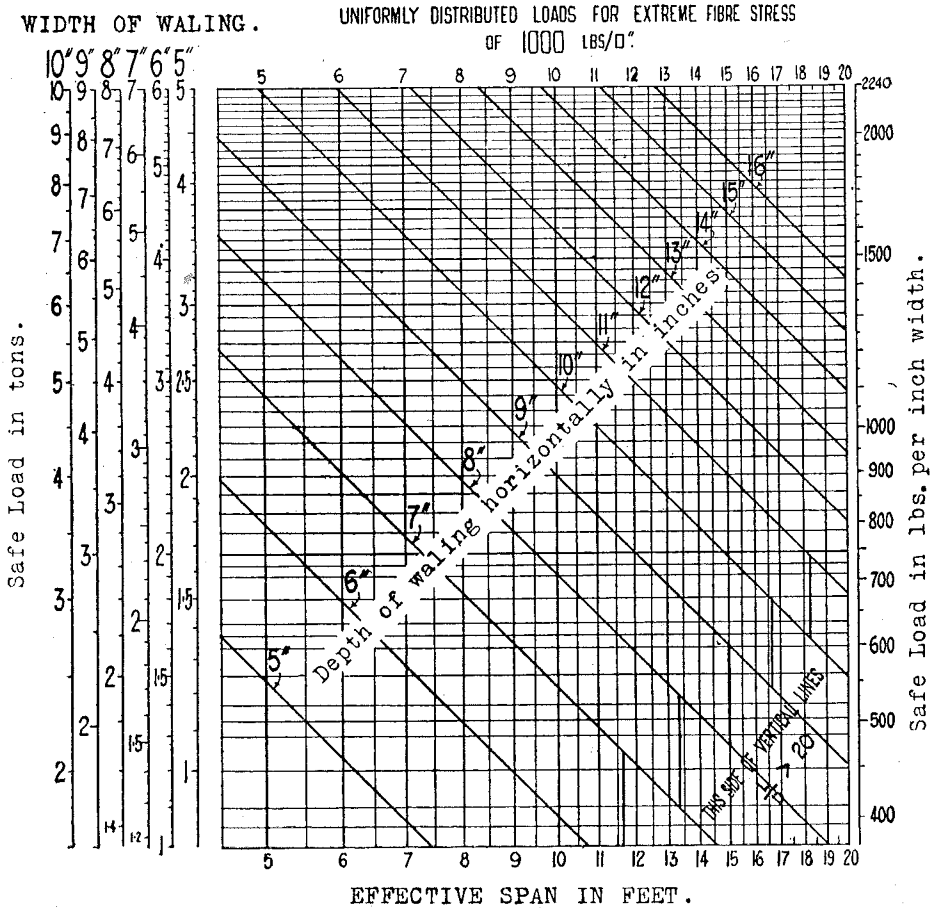


FIG. 70.—GRAPH FOR THE DESIGN OF TIMBER WALINGS.

based upon a stress of 1,000 lb. per square inch, so that the safe load for any 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. For uniformly distributed loads and disregarding continuity, the maximum safe load on a beam or waling will be 4/3 times the maximum safe horizontal shear stress. Thus 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. When making a trial design it may be assumed that the maximum safe load on a beam is 160 lb. times the cross sectional area in inches, but for 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 maximum span that the sheet piling will withstand (*Fig.* 65). A suitable proportion of the maximum span given in *Fig.* 65, 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.* 65, 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

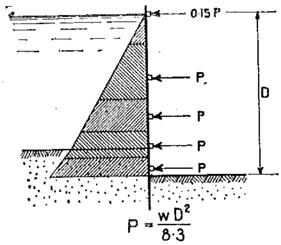


FIG. 71.—APPROXIMATE LATERAL LOAD ON WALINGS SPACED AS IN FIG. 65.

are continuous the reactions at these intermediate supports will be greater and the deflection of the walings less than if they were in short lengths. This may, however, generally be disregarded and, 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 is the shaded area in *Fig.* 71, which, assuming full continuity of the sheet piling over at least five levels of walings, gives a horizontal load per linear foot of $\frac{wD^2}{8{\cdot}3}$, or slightly more for shallower cofferdams. The waling will have axial compression because it also supports the end of the second waling across the end of the dam, and in the case of bracing forming squares in the plan of the interior the axial load through the length of the waling will be approximately half the transverse load on the walings.

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 if the axial compression stress intensity in the waling is found roughly by finding the approximate size of the waling from *Fig.* 70, the suitable section is obtained by deducting the axial compression stress from the permissible bending stress for the particular grade of timber being used and using this reduced stress in *Fig.* 70.

For the transverse struts the accidental transverse impact load could perhaps reasonably be taken as 1,000 lb., in comparison with which the bending stress by self weight is relatively insignificant and may be ignored.

First find a square section timber that will take the waling reaction at a stress of about 800 lb. per square inch and, using the modulus of this section, find the bending stress that results from the accidental impact point load at the centre of the unsupported length of the strut. Then deduct one-and-a-half times this bending stress from the permissible axial compressive stress (say, from *Table* VIII) in the strut, and determine the size of the strut for axial stress on this reduced stress (*Table* IX).

TABLE IX.

SAFE LOAD, IN POUNDS, ON SQUARE TIMBER STRUTS.

For Southern Yellow Pine, occasionally Wet.

Side of square (inches)	8	10	12	14	16
Maximum safe load for short strut	68,200	106,500	153,400	208,700	272,600
Length (ft.)					
7	67,800				
8	66,900				
9	66,000	105,700			
10	65,000	104,500	153,400		
11	63,600	103,500	151,900		
12	62,000	102,300	150,500	208,200	
14	56,300	98,900	148,000	204,800	271,400
16	48,600	93,600	144,400	202,100	267,500
18	38,700	86,200	139,400	198,200	264,200
20	31,200	75,900	131,800	192,900	259,800
22	25,900	63,100	122,300	185,000	254,500
24	21,600	53,100	109,300	175,400	247,800

Maximum allowable compression parallel to grain, 1,065 lb. per square inch.

For struts having a length more than ten times the lesser side of the cross section the stress to be used should be reduced for greater slenderness, for example, according to the stress curve shown in *Fig.* 72 * which is based upon the London County Council By-laws, 1935.

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, a concrete seal is usually placed which may itself be the foundation concrete for the work being constructed,

* The number of the stress grade is the working stress in lb. per square inch which may safely be used for that particular quality of timber, and the subsequent letter *f* or *c* differentiates between stress grades applicable to beams and columns. It is to be noted that the limiting defects for grades of timber for beams and columns differ; for example, a quality of timber that might be graded 1600*f* for use as a beam might be graded 1200*c* for use as a strut.

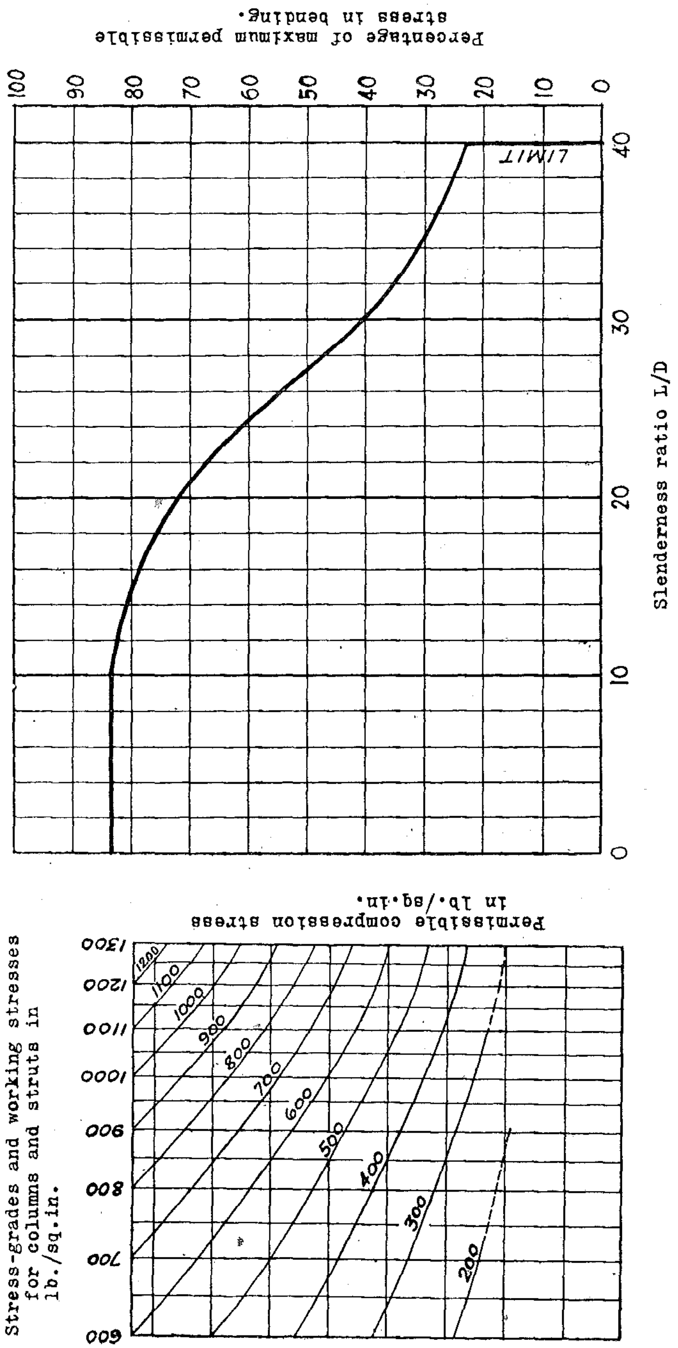

Percentage of maximum permissible stress in bending.
0 10 20 30 40 50 60 70 80 90 100
LIMIT
0 10 20 30 40
Slenderness ratio L/D
Permissible compression stress in lb./sq.in.
1200 1100 1000 900 800 700 600 500 400 300 200
Stress-grades and working stresses for columns and struts in lb./sq.in.
600 700 800 900 1000 1100 1200 1300

FIG. 72.

or may be merely a concrete sealing layer to enable the foundation to be constructed under comparatively dry conditions.

In the latter event the concrete seal may be taken right up to the steel sheet piling forming the wall of the cofferdam. If the steel sheeting is to be subsequently removed, it is best to drive each sheet pile an inch or so, while the concrete is hardening, so as to break the bond and facilitate subsequent extraction.

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 wellpoint system is suited to this method, as a number of wellpoints, 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 wellpoints subsequently disconnected and sealed off.

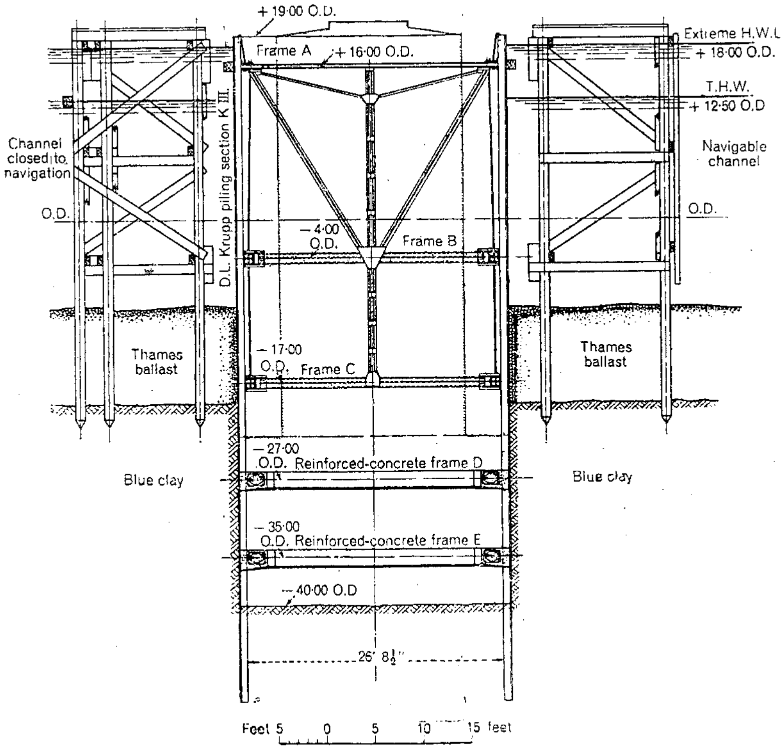
An alternative method with ordinary pumping is to lay open-jointed drainage either in pipes or coarse material, and to have one or more sumps into which the foot-valves and strainers of the pumps are buried in rubble or hardcore. In this case success will depend upon the drainage channels not becoming blocked with fine material before the concrete seal has been placed and hardened. The concrete seal has to be placed under water if the upward flow is considerable, and generally the wellpoint 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.

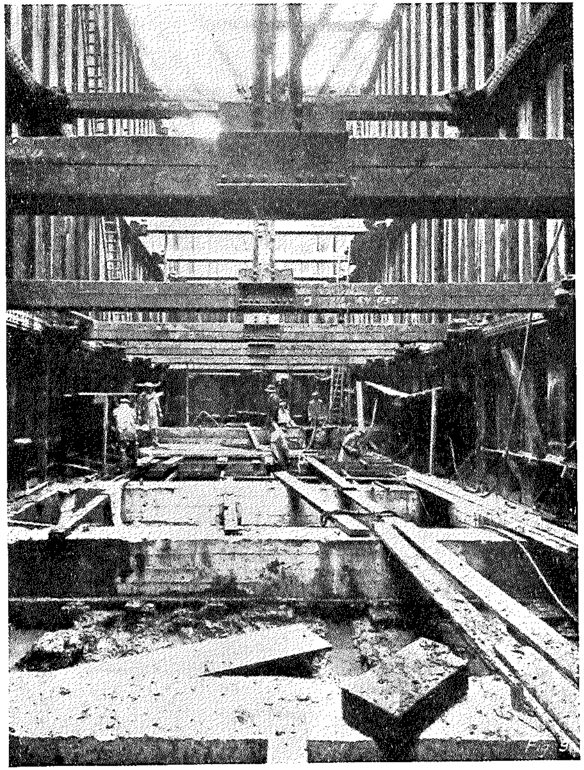
To maintain the concrete seal in position undisturbed while further load is added will usually necessitate allowing the cofferdam to fill, or filling it by pumping if for any reason the pumping through relief pipes is interrupted by a breakdown.

Examples of Cofferdams.

As an example of the increased use of internal strutting of structural steelwork and the improved working space provided, *Figs.* 73 and 74 show a combination of structural steel and reinforced concrete walings and struts used for the cofferdam for a pier of Chelsea Bridge.(35) 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,(36) 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 trusses were provided to bridge the subway





FIGS. 73 AND 74.—COFFERDAM FOR A PIER OF CHELSEA BRIDGE, LONDON.

and were lowered into each cofferdam (*Figs.* 75 and 76). The two top stays of bracing were fabricated as steel trusses and dropped into place as a unit. After

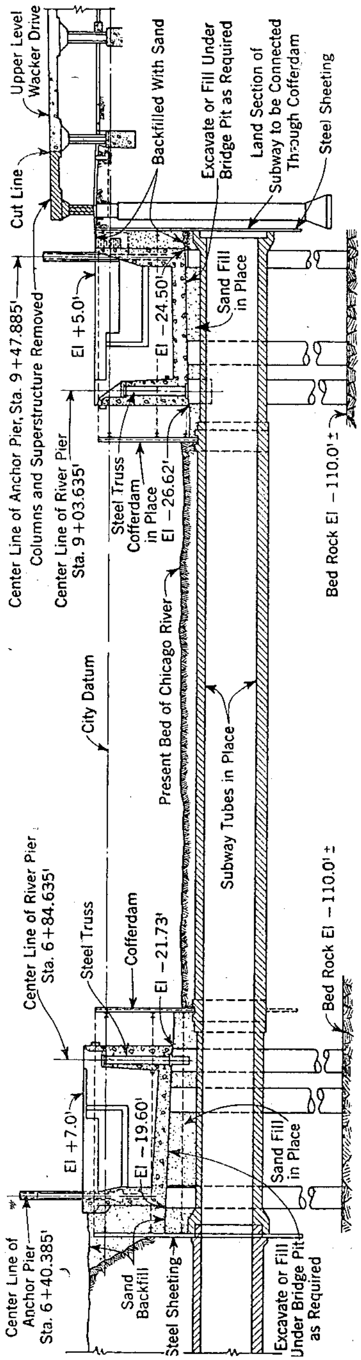


FIG. 75.—COFFERDAM FOR A PIER OF NORTH STATE STREET BRIDGE, CHICAGO.

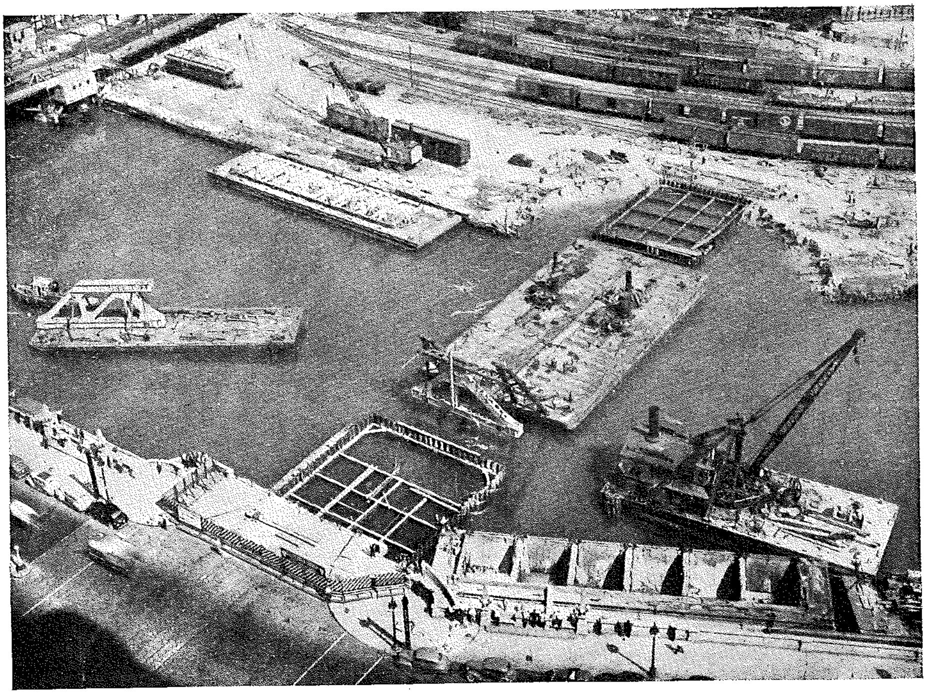


FIG. 76.—COFFERDAM ILLUSTRATED IN FIG. 75 UNDER CONSTRUCTION.

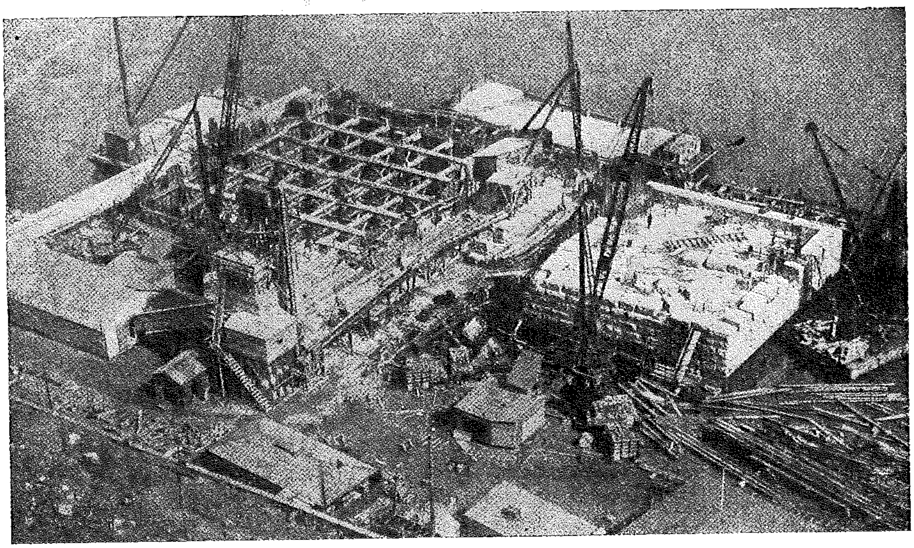


FIG. 77.—COFFERDAM FOR A PIER OF THE NEW JERSEY TOWER, GEORGE WASHINGTON BRIDGE.

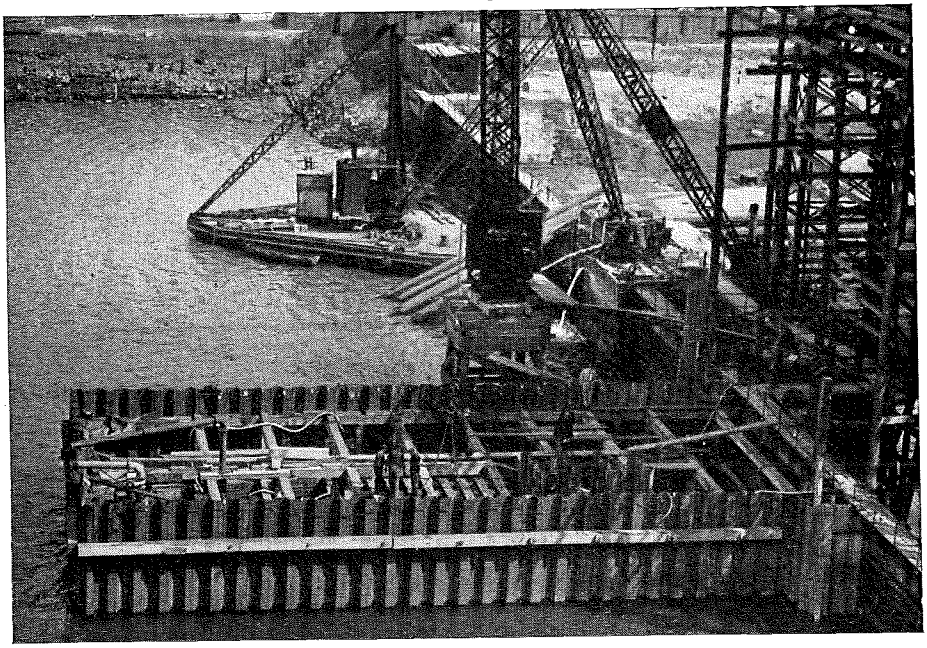


FIG. 78.—STEEL SHEET-PILED COFFERDAM WITH TIMBER BRACING.

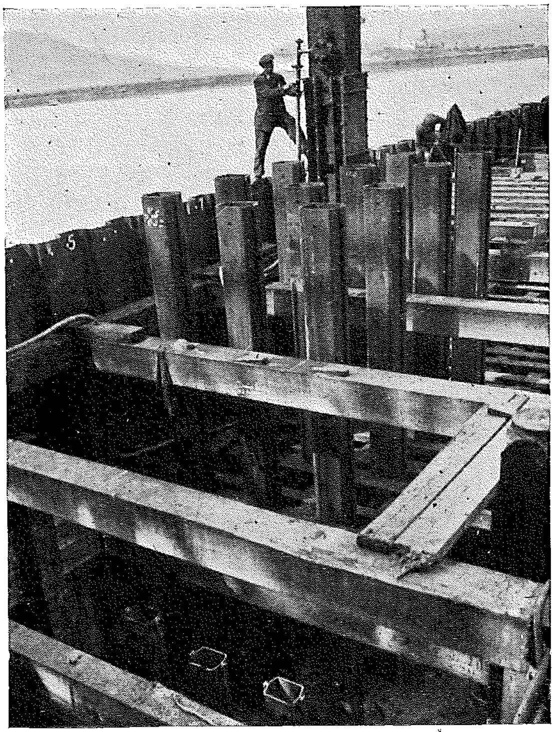


FIG. 79.—DRIVING STEEL-SHEET BOX PILES INSIDE COFFERDAM ILLUSTRATED IN FIG. 78.

de-watering, 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-piled cofferdams with timber strutting is illustrated

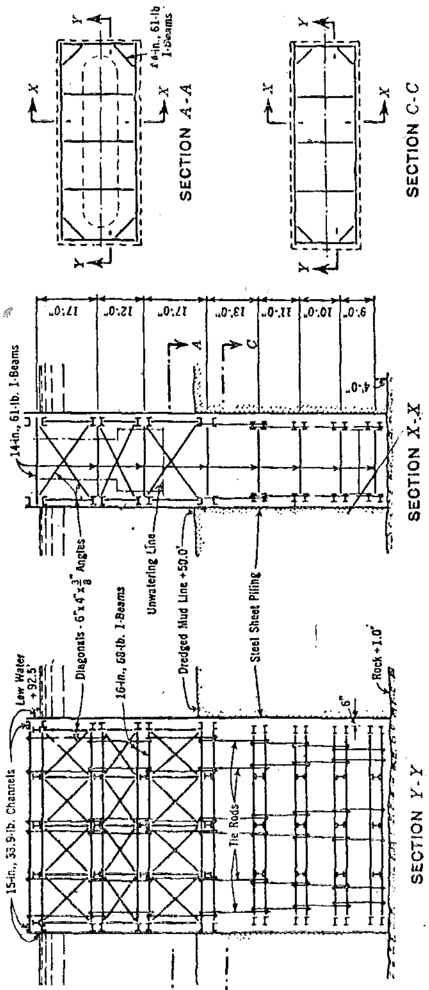


Fig. 80.—Cofferdam for a Pier of the Lake Champain Bridge.

in *Fig.* 77, which shows the foundations under construction for the New Jersey tower of the George Washington bridge.[37]

Another example of steel sheet piled cofferdam with timber internal bracing is shown in *Figs.* 78 and 79. 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.* 79.

For the channel piers for the Lake Champlain bridge structural steel bracing was used (*Fig.* 80). The contractor's plan [38] 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 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.* 81. 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 (e.g. 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; an error on the safe side is obtained in this being assumed to balance secondary moments in the vertical plane, due to unavoidable discrepancies in alignment of the steel ribs when supported by brackets from the piling.

Cellular Cofferdams.

Whereas the sheet piling previously described has been stressed in bending, with types of steel piling having strong interlocks it is possible to use cofferdams of cellular construction in which the piles are stressed transversely in tension. 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.* 55, the type utilising complete circles with short connecting arcs, of which an example is shown in *Fig.* 82, is the better all-round construction.

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

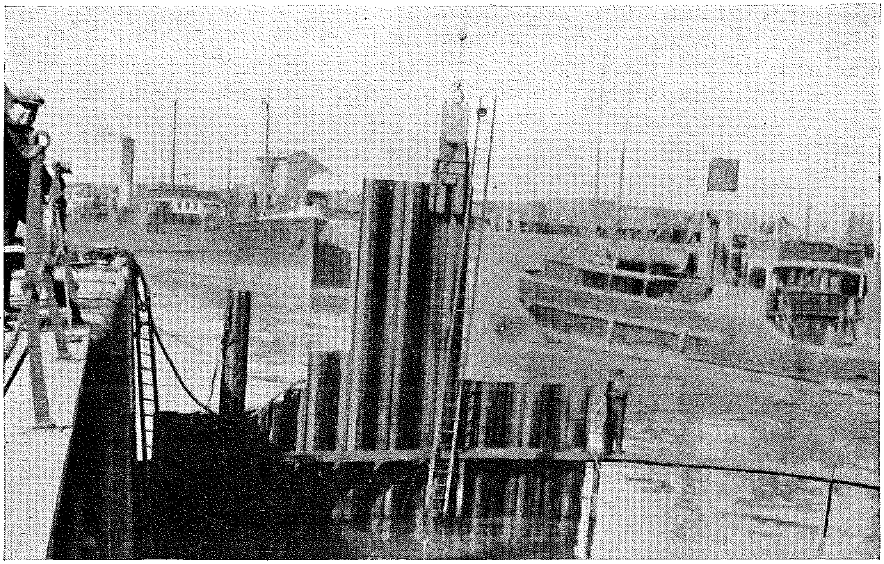


FIG. 81.—ARCHED COFFERDAM FOR ENTRANCE OF WHITEHAVEN HARBOUR DOCK DURING CONSTRUCTION.

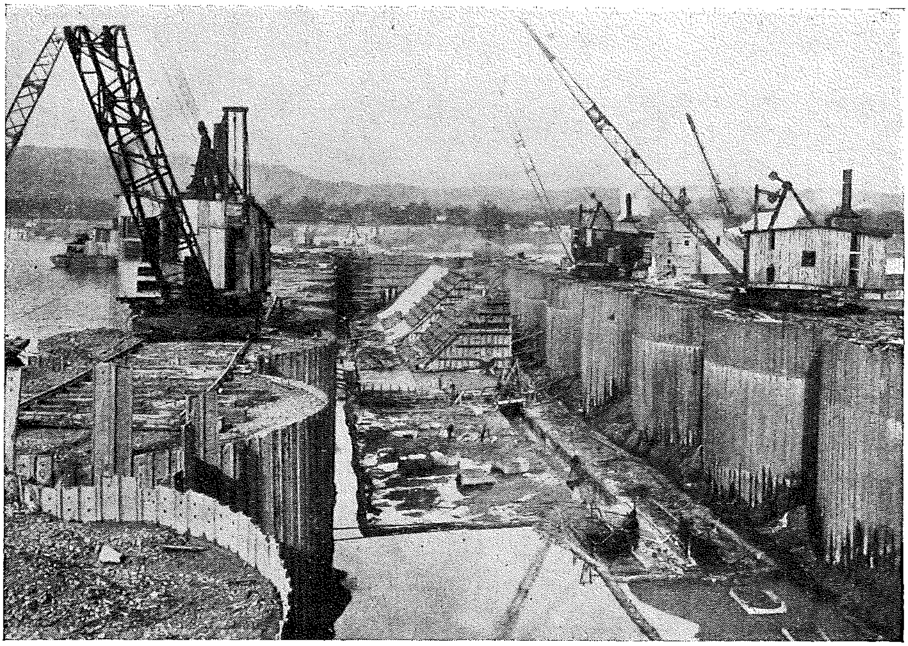


FIG. 82.—CELLULAR COFFERDAMS.

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, in the writer's opinion, 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 recent practice in America, where sections such as M107 and M108 (see *Table* II) are available. For these particular sections the strength of the interlock in tension is 12,000 lb. per linear inch of interlock, while for sections M106 and M117 it is 10,000 lb.

Deeply corrugated sections are generally not suitable for this type of use.

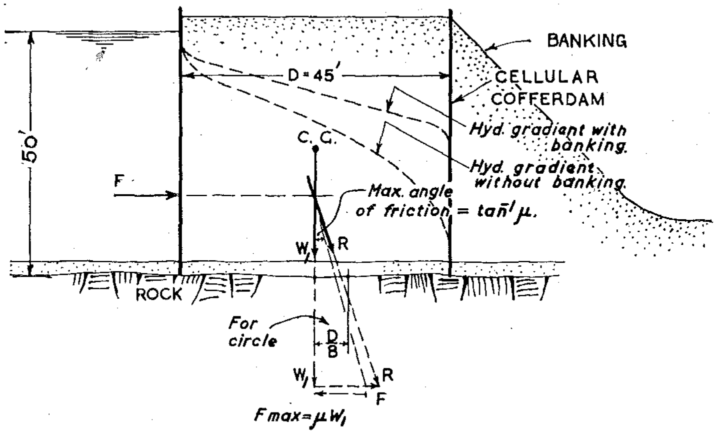


FIG. 83.—STABILITY OF CELLULAR COFFERDAMS.

Referring particularly to the circular cell type of cofferdam (*Fig.* 82), 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.

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 that d is the required diameter of the cell, then, ignoring the fill between the short arcs connecting the circles, the necessary diameter of the cell is best obtained from the relation with the stability of a length d of solid fill. If the width of fill (l) is found in the usual manner the diameter d of the cell of the same stability is given by $d = \sqrt{\frac{4l^2}{2 \cdot 356}}$.

The resultant R (*Fig.* 83) 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 unfavourable 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 piled walls or due to similarity with the conditions applying in the design of bins, by which arching is taken into account in formulæ such as those of Jannsen and Airey, 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.

When filling cells with pumped 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.* 83, 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 of steel piling should preferably be not less than $2\frac{1}{2}$.

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, $= 0{\cdot}42wDH/2$ where H is the height of the soil fill, D is the diameter of the cell, and w is the density of the soil, dry or 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

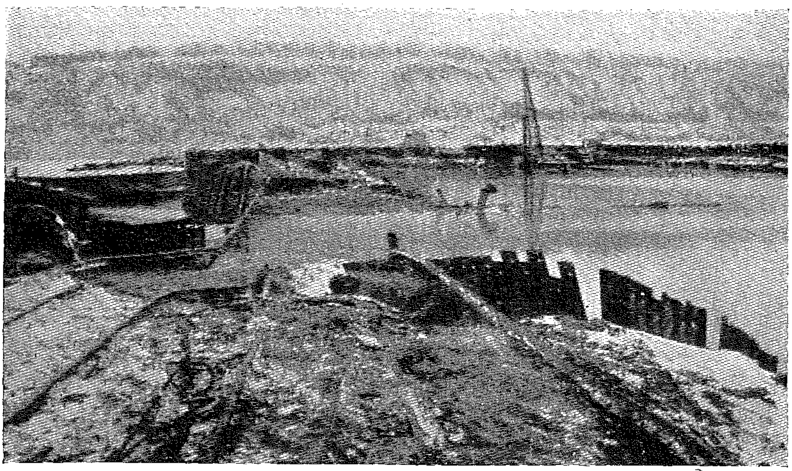


FIG. 84.—FAILURE OF DIAPHRAGM TYPE CELLULAR COFFERDAM.

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, as can be seen in *Fig.* 82.

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 approximately half 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.

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.* 84.

The usual method of filling cellular sheet piled bulkheads by pumping dredged material is shown in *Fig.* 85. In this case the diaphragm-type steel sheet piled

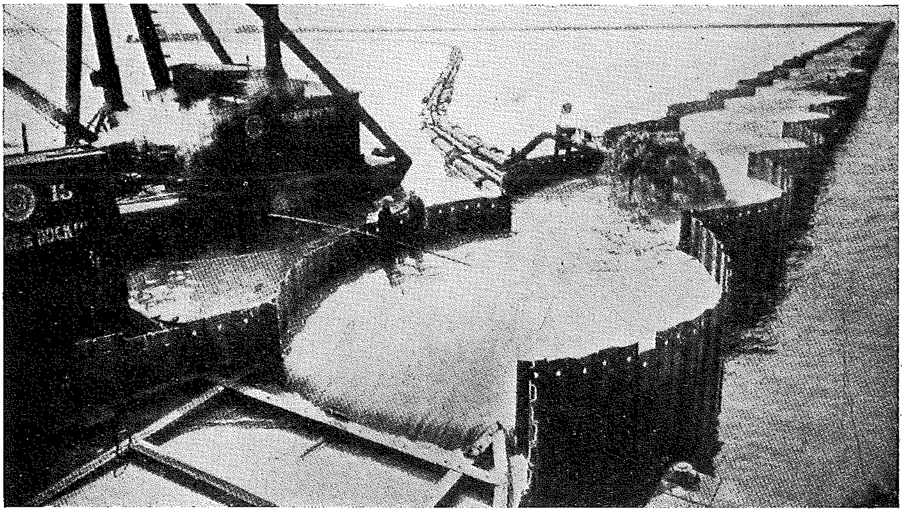


FIG. 85.—FILLING DIAPHRAGM TYPE CELLULAR BULKHEAD WITH DREDGED SAND.

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. The section of steel pile used was that shown in *Fig.* 86 and has an interlock strength in direct tension of 12,000 lb. per lineal inch.

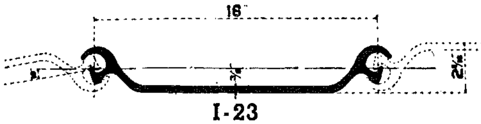


FIG. 86.—SECTION OF STEEL SHEET PILE SHOWN IN FIG. 85.

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.[39] From *Table* X 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 cofferdam stages, and similar 15-in. piling with $\frac{3}{8}$-in. web was used on the three sites. 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

TABLE X.

DRIVING PERFORMANCE ON LOCK COFFERDAMS FOR T.V.A. PROJECTS.

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

* 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.

FIG. 87.—COFFERDAM FOR PIER OF WATERLOO BRIDGE, LONDON.

at the Pickwick Landing dam. The average over the three cofferdam stages of that site was

	Dollars per ton
Handling and hauling	1·405
Templates	2·541
Setting piling	5·602
Driving	3·639
Pile pulling	6·279

Examples of cofferdams for bridge foundations (*Figs.* 87 and 88) show the reconstruction of Waterloo Bridge and a bridge at Baghdad.

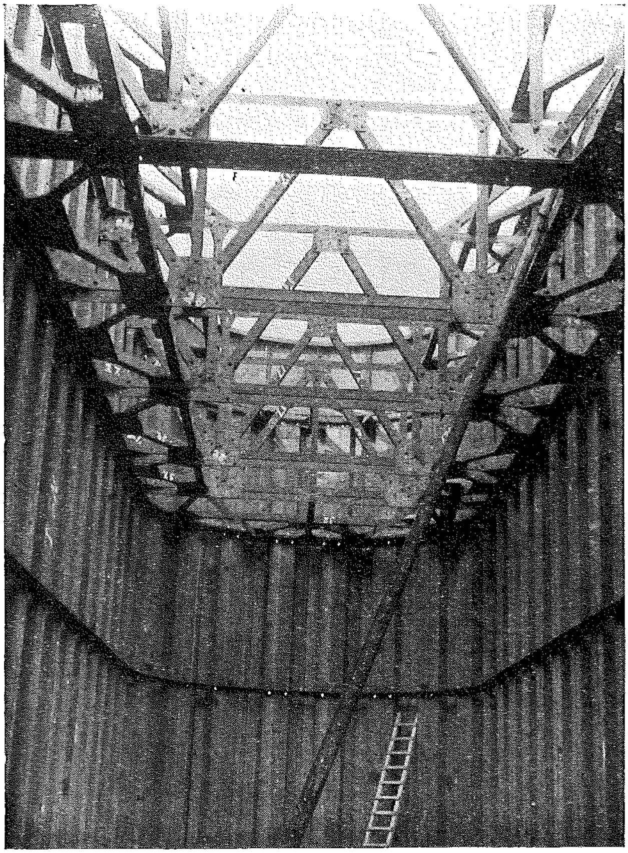


FIG. 88.—COFFERDAM FOR PIER OF BRIDGE IN BAGHDAD.

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 [40] in Denmark, an alternative method (*Fig.* 89) 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 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.

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 as shown in *Fig.* 90, 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

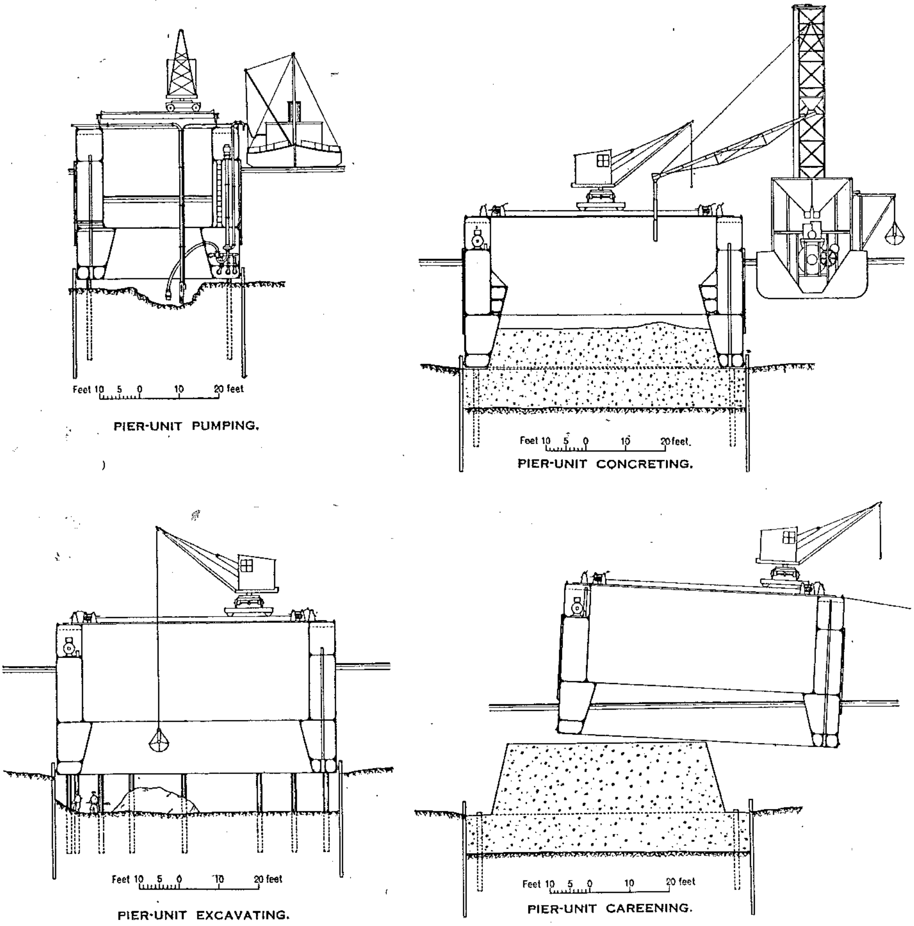


FIG. 89.—METHOD OF USING MOVABLE COFFERDAMS, STORSTROM BRIDGE, DENMARK.

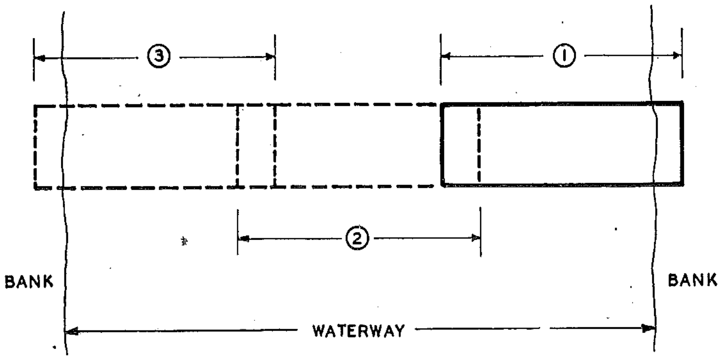


FIG. 90.—COFFERDAM STAGES IN A FLOWING WATERWAY.

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 de-watering 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.

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.

Cofferdams 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. 91 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 [41] regarding the cofferdam construction :

" It was intended to construct each season's sudd [*Figs.* 91 and 92] 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 top level of the piles was R.L. 64.50 for the first season, being reduced to R.L. 63.50 for the second season and to R.L. 63.00 for the third season.

" Two floating pontoons, each fitted with travelling leaders allowing ten piles to be pitched and driven before a pontoon was moved farther along the line, assisted occasionally by a 40-ft. piling frame mounted on the bow of a wooden barge, were employed in driving the sudd piles, Nos. 6 and 7 McKiernan-Terry hammers being used.

" The sudd enclosing the area of the first season's work was in the form of a truncated triangle with its base on the west bank of the river. A start was made at the south-west corner of the sudd, and the first pile was driven on the 7th November, 1927, but owing to the rapid fall of the river the pontoon had to be moved to the south-east corner to work in a northerly and north-westerly direction, where there was sufficient depth of water. The second piling-pontoon was set to work on the 5th December, and piling was continued until the end of December, by which date

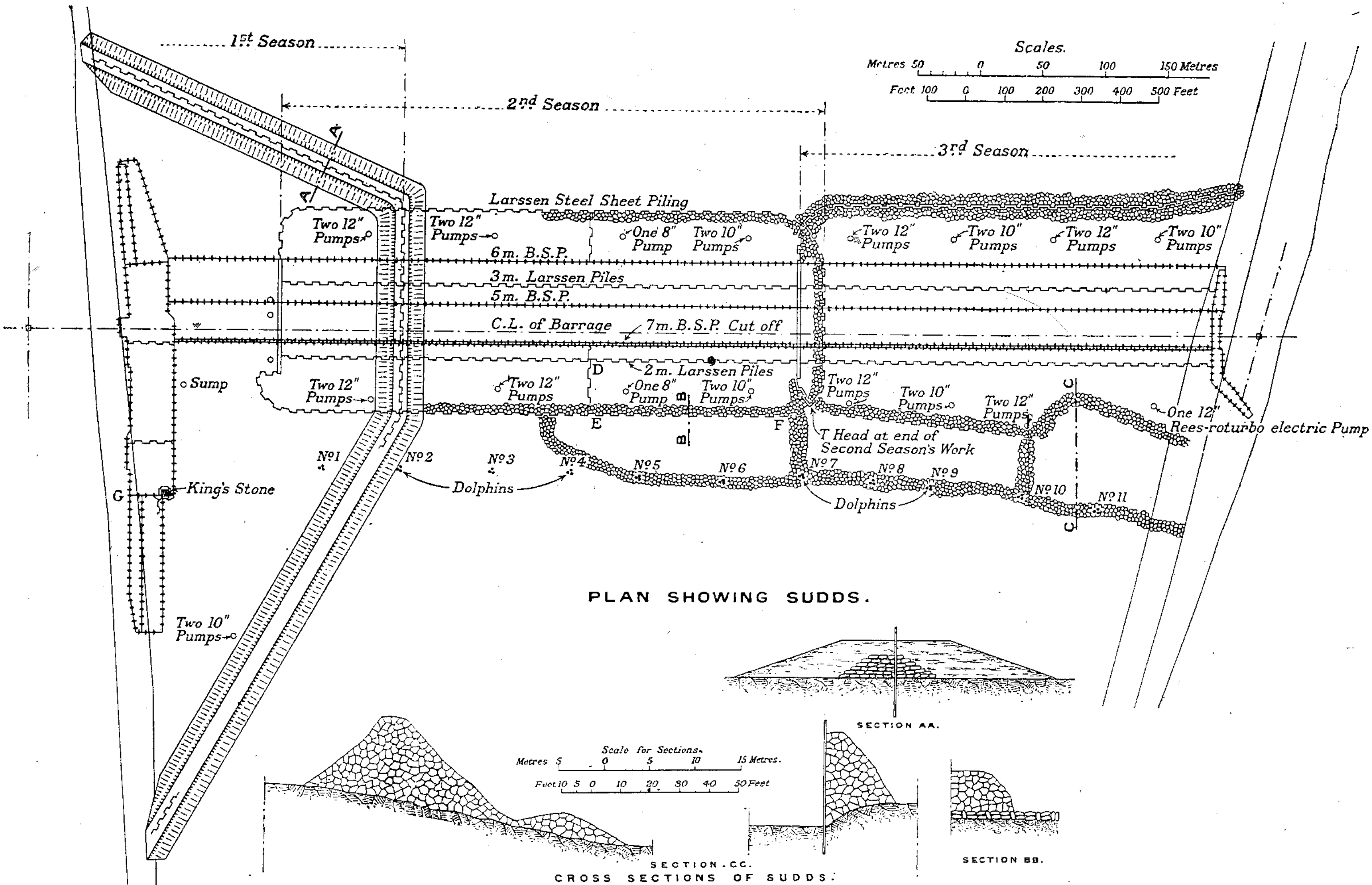


Fig. 91.—The Nag Hammadi Barrage on the River Nile. [(The north (downstream) side is at the top of the plan.)]

1,058 piles, forming the eastern and north-western arms of the sudd, had been completed. Meanwhile the southern diagonal had been built with sand-bags and tipped excavation.

" The contractors also drove two Larssen sheet-pile diaphragms across the lock foundation and two similar diaphragms across the barrage foundation, one on the centre-line of the tenth pier and one on the centre-line of the twentieth pier. These diaphragms divided the foundations into convenient pumping areas, and in the case of the barrage they acted as retaining walls to support the sand when the foundations were stepped up to a higher level."

With regard to the construction of the sudd for the second season, the following extract is of interest with regard to scour:

" Very soon after pile driving had been started it was discovered that the water flowing past the leading piles was scouring the bed of the river to such an extent

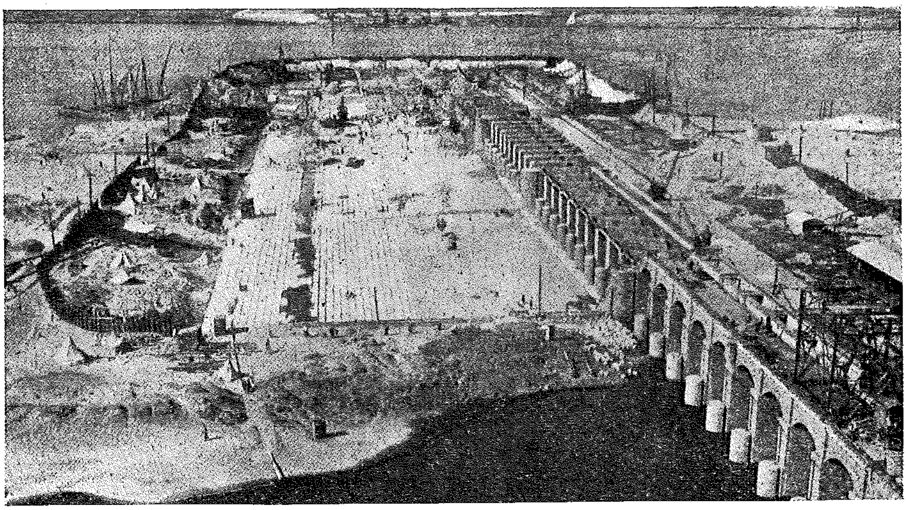


FIG. 92.—CONSTRUCTION OF SUDDS, NAG HAMMADI BARRAGE.

that there was no hold for the piles. The high velocity in the eastern channel was due to the fact that the first ten vents were partly masked by sand-banks created by depositing some of the first season's excavation upstream and by the remnant of the first season's sudd. Sand-bags were tipped alongside and ahead of the piles, but proved to be ineffective. A rubble-stone bar was therefore tipped on a line parallel to the southern side of the sudd, and 50 metres upstream, until its crest was brought up to water level, the leading end of the bar being kept well ahead of the leading end of the southern row of piles. The sudd was closed on the 5th January, 1929, and unwatering was completed on the 22nd January. In 75 days of 24 hours 2,783 piles were driven, the largest number of piles driven by one machine in one day being 78."

Owing to scour occasioned by the increased velocity of current outside the sheet piling, one of the sudds developed a boil which very quickly resulted in the flooding of the entire interior. Possibly the blow might have been prevented if the velocity of the current past the outside of the sheet piling had not made soundings very difficult.

It is of interest also that on this contract better penetration was obtained for the cut-off piles by using drop hammers, but, as this method resulted in more damage to the tops of the sheet piles, double-acting hammers were used except where the tops of the sheet piles were subsequently embedded in concrete.

Remodelling of the Assiut Barrage.

This work was carried out during four low-water seasons between the years 1934 and 1938, and consisted essentially of raising the original barrage to take a maximum head of 17 ft., involving, besides the new masonry, sluice gates, lift bridges and machinery, the improving of the apron slabs by widening both upstream and downstream. The total amount of the contract was £E1,140,000, and involved, inter alia, some 181,000 cubic metres of excavation, 2,033 reinforced concrete piles, and 1,442 tons of steel piles.

The total width of the Nile at this point is about 2,700 ft. *Fig.* 93 shows a

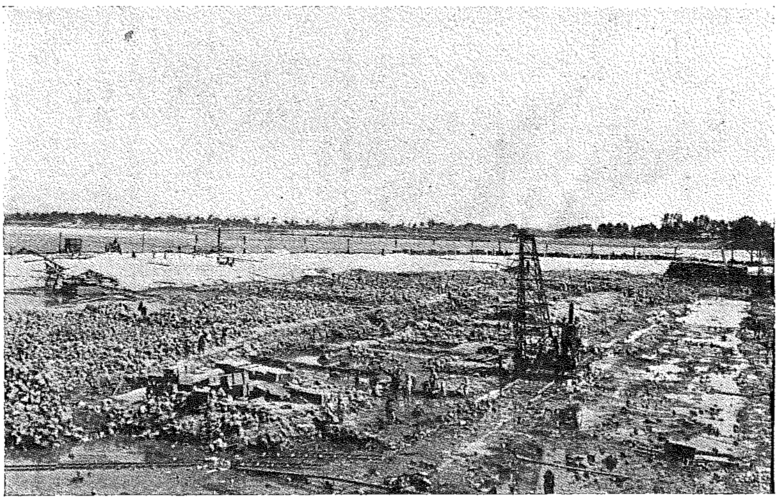
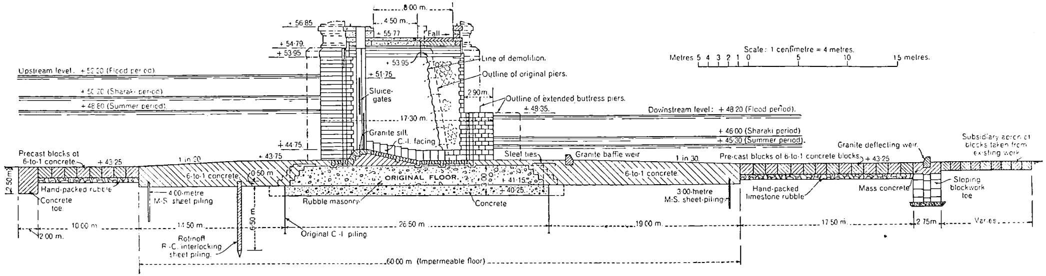


FIG. 95.—RECONSTRUCTION OF THE ASSIUT BARRAGE, SHOWING SUDD.

cross-section through the barrage and *Fig.* 94 gives cross sections through the upstream and downstream sudds. A general view of the work is seen in *Fig.* 95. 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 [42] 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.



TYPICAL CROSS SECTION OF BARRAGE.

FIG. 93.—CUT-OFF SHEET PILING AT THE ASSIUT BARRAGE, AS RECONSTRUCTED.

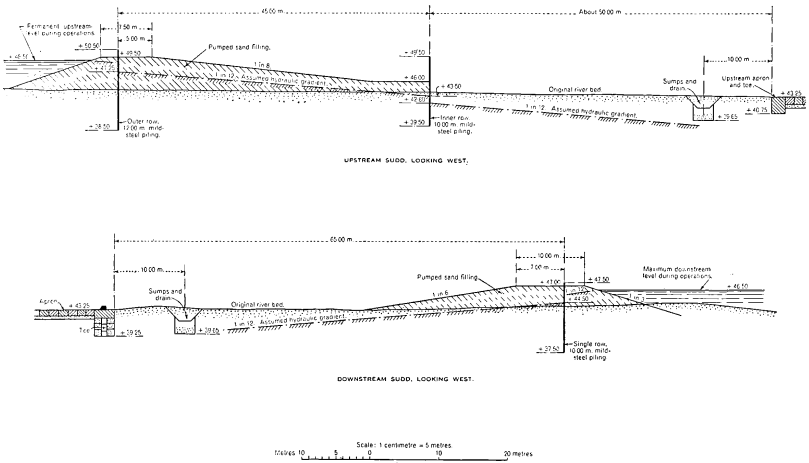


FIG. 94.—TYPICAL SECTION OF SUDDS, ASSIUT BARRAGE.

"(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."

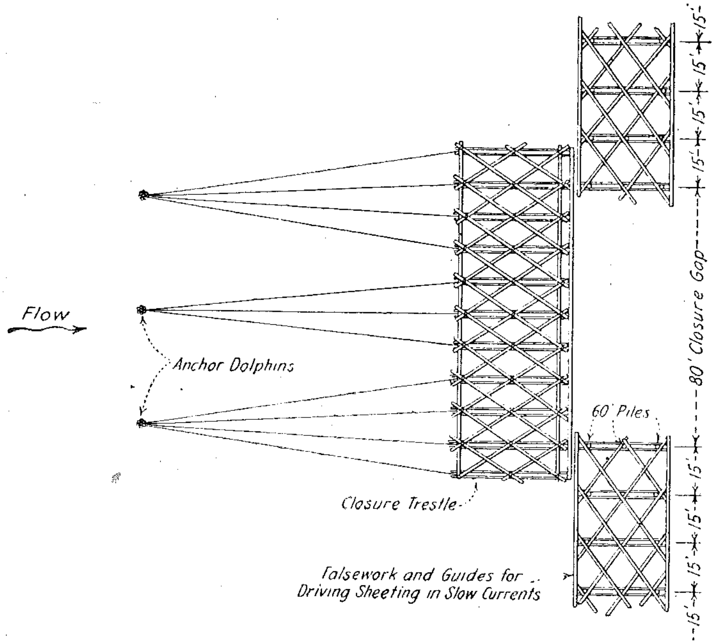
With regard to the piling for the sudds, which was Larssen No. 2 driven by McKiernan-Terry hammers of sizes Nos. 6 and 7, and exceptionally No. 9, the piling was driven to a depth of roughly 10 metres below the river level and about 5 metres into the river bed. On the downstream side a single row of steel piling was driven to a similar depth below the river level. At the forward end of the 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 3,720 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 de-watering was carried out slowly in order to allow the sudds to drain out gradually, after which there was little seepage water throughout the sudds.

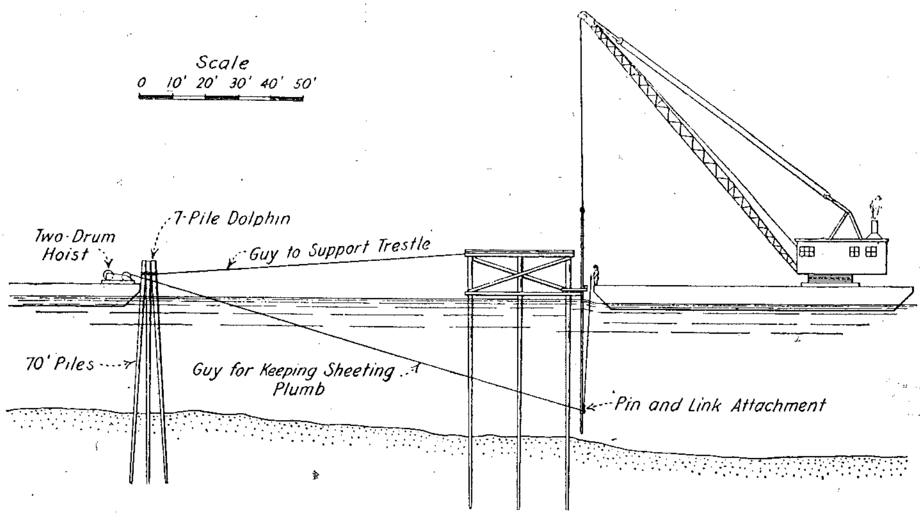
Closure Operations in Rivers.

The writer is indebted to Messrs. White and Prentis for *Fig.* 96 showing the method used to maintain sheet piling plumb during closure operations for dam No. 26 on the Mississippi river at Alton, Illinois. The general nature of this operation can be seen also in *Fig.* 97, and complete details of the work are given in their book on cofferdams,(33) 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.

In effecting the closure indicated in *Fig.* 96, use was made of three dolphins each of seven 70-ft. piles placed 100 ft. on the upstream side of the closure, with a ¾-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



PLAN OF CLOSURE TRESTLE AND FALSEWORK GUIDE.



METHOD OF SETTING STEEL SHEET PILING IN SWIFT WATER.

FIG. 96.—METHOD OF EFFECTING CLOSURE OF FINAL COFFERDAM STAGE.

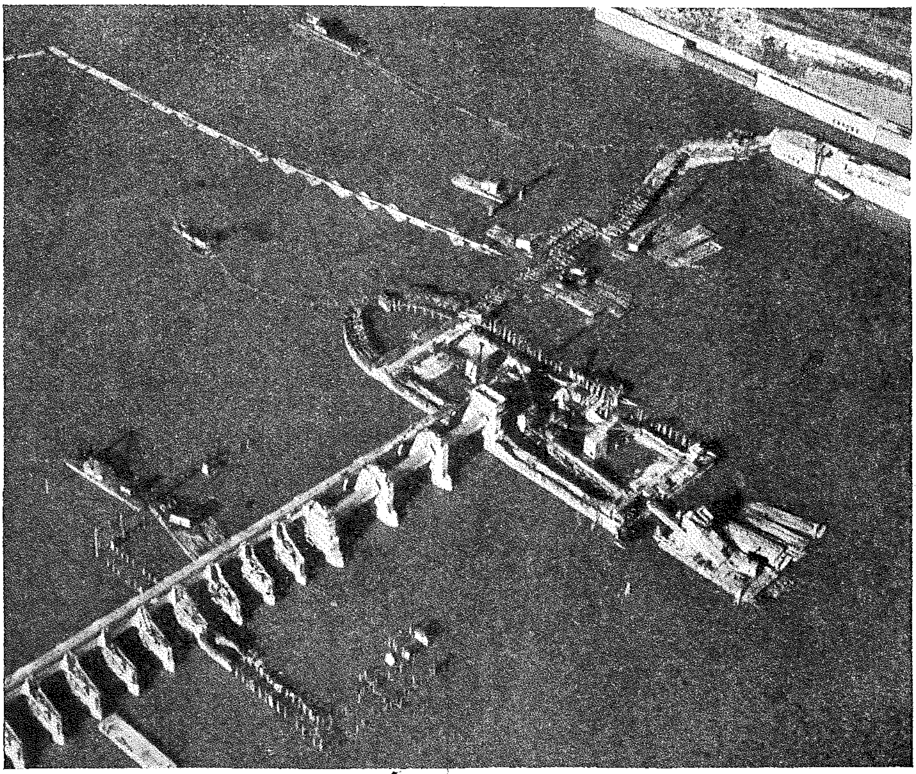


FIG. 97.—CLOSURE OPERATIONS FOR FINAL COFFERDAM STAGE, LOCK AND DAM NO. 26, UPPER MISSISSIPPI RIVER.

(See upper part of illustration and also Fig. 96.)

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.

PART IV

CYLINDERS AND CAISSONS: GENERAL AND THEORY

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 de-watered, 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, or where the total load is large; also in cases where it is spread over a reasonable area, and if in either case the depth to the bearing soil does not exceed 60 ft., or in special cases 90 ft. Also, the cofferdam method is usually 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.

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.* 98, 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

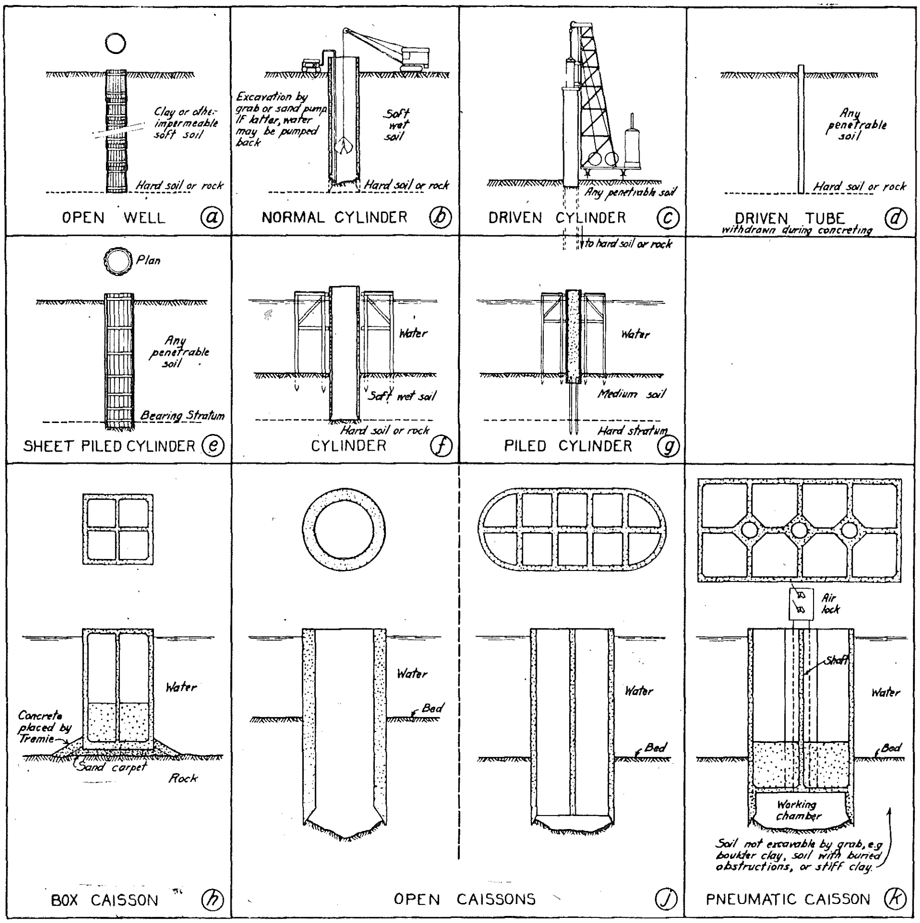


FIG. 98.—EXAMPLES OF CYLINDERS, CAISSONS, AND COMPARABLE METHODS.

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.* 99, 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.

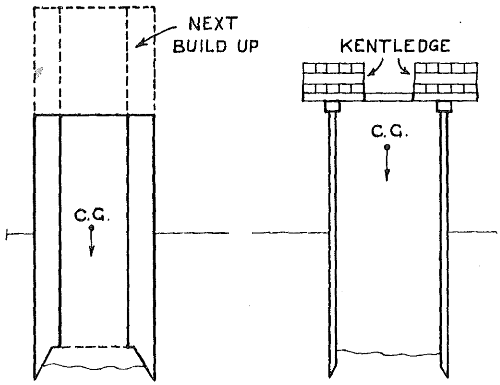


FIG. 99.—SHOWING THE GREATER STABILITY OF CAISSONS COMPARED WITH CYLINDERS.

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 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.

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 wellpoint system of pumping), then box caissons [*Fig.* 98 (*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.* 98 (*f*)], or alternatively [*Fig.* 98 (*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.

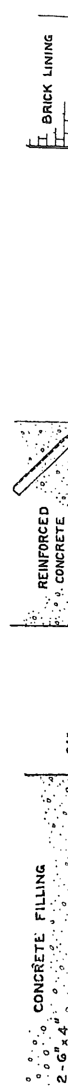
Open Caissons.—The method of sinking an open caisson and grabbing out the soil under water is generally the same as the cylinder method, except that with caissons the walls themselves carry part or all of the load, sometimes by filling compartments with concrete or masonry as sinking proceeds, and open 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 these types will be described later to show site circumstances that 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 de-watered successfully the concrete may be dropped from the surface from concrete skips; on the other hand, if it is not practical to de-water 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 167.)

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.* 100 (*e*) may be suitable in which the seal supports a cylinder or caisson by a horizontal surface of contact, and this type of con-

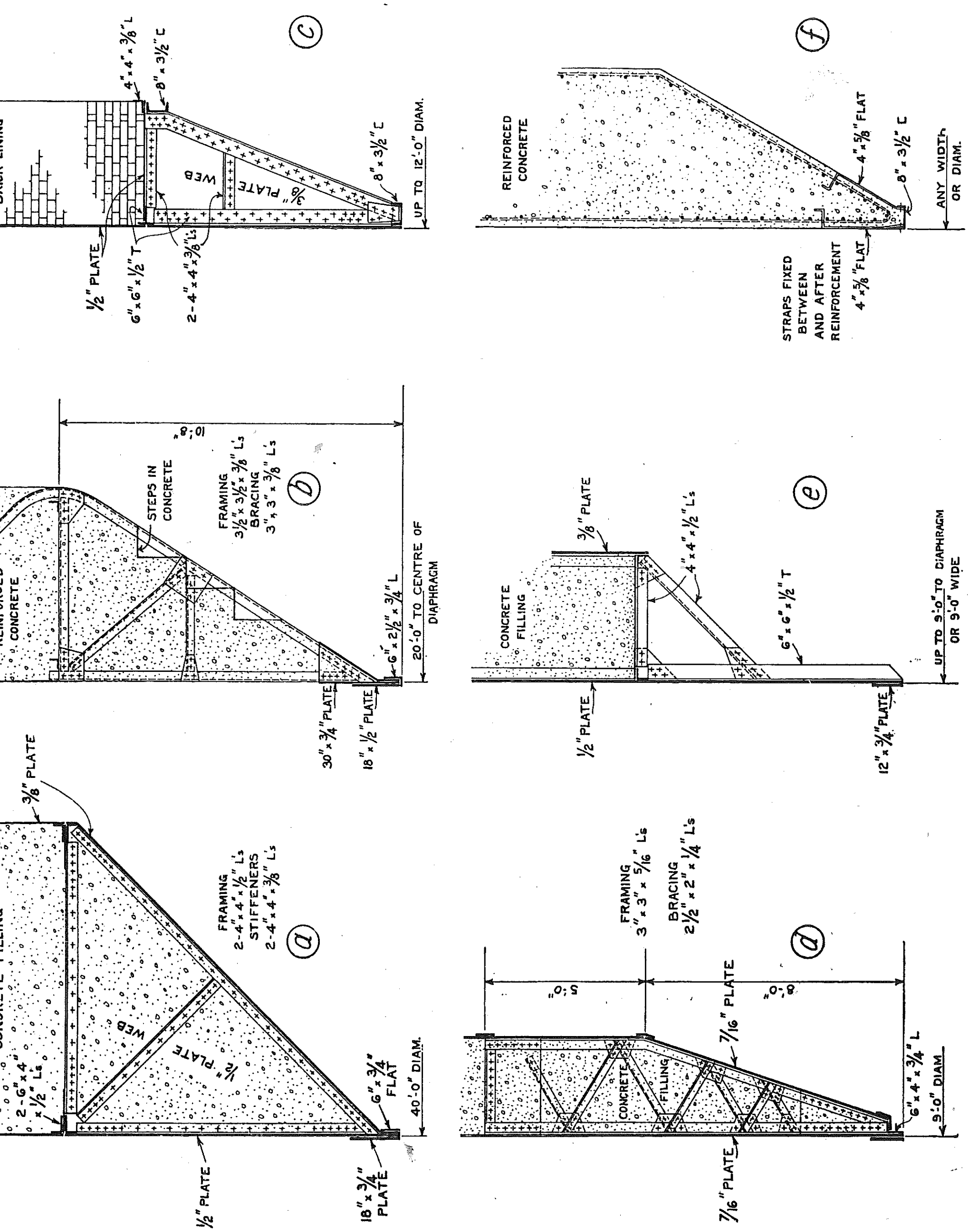


FIG. 100.—EXAMPLES OF CUTTING-EDGE DETAILS.

struction has been used for pneumatic caissons. The detail (*a*) is typical of earlier practice for open cylinders of steel construction, for example, that used for the caissons of the Forth Bridge was of this type. 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,[43] 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 strakes 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,[44] 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 comparatively large caissons have been constructed without cross walls, and this may occasionally still be practical and economical to-day in the case of small depths. More 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 assumed to 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.* 101.

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 shears and 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 will be provided by the skin friction of the soil

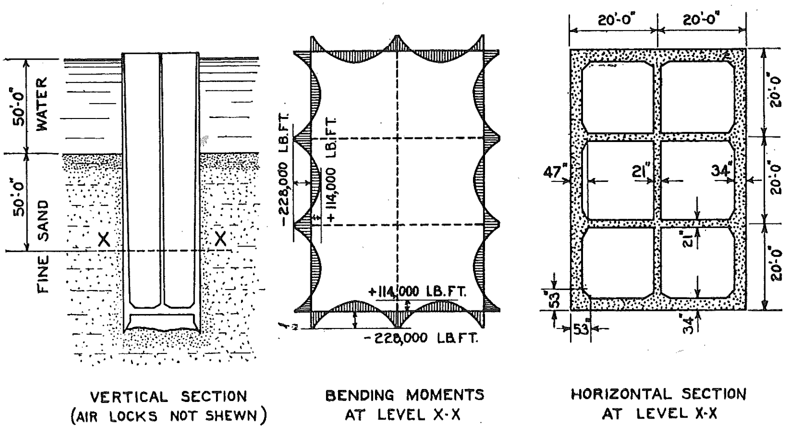


FIG. 101.—EXAMPLE OF BENDING MOMENTS IN CAISSON AT A DEPTH OF 100 FT. BELOW WATER SURFACE.

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 inconceivable 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.

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

$$F = \frac{\mu w h^2}{2}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right),$$

that is to say, using the Rankine formula for the active pressure for the depth, in conjunction with the coefficient of friction (μ) for the particular soils penetrated.

For granular soils only it can be seen from *Fig.* 102 that an apparent value

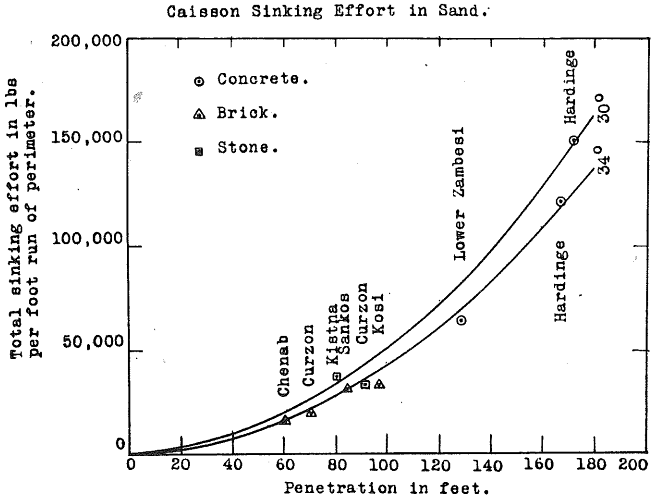


FIG. 102.—RELATION OF SINKING EFFORT IN SAND TO RANKINE VALUES OF THE ACTIVE PRESSURE WITH COEFFICIENT OF FRICTION (μ) = 0·3.

of μ 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.* 103 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

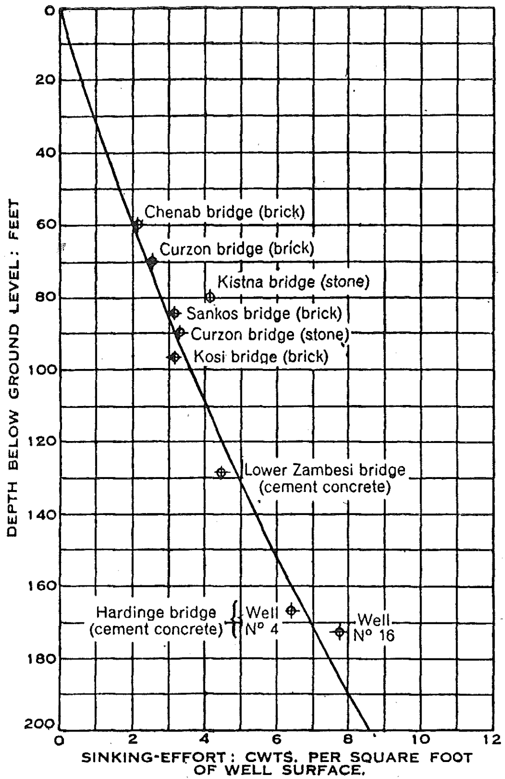


FIG. 103.—SINKING EFFORT DIAGRAM FOR OPEN CAISSONS SUNK IN SAND.

then some trouble was experienced in reaching the required depths, and serious blows occurred before the caissons reached their final levels.

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 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* XI, mostly obtained from data collected by H. L. Wiley,[45] 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 [46] 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.

Compressed-Air Working.

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

TABLE XI.

SKIN FRICTION IN SINKING CYLINDERS.

No.	Type of caisson	Method of sinking	Materials penetrated	Average skin friction (lb. per sq. feet)	Depth below low water in feet	Area of base in square feet
			MAINLY GRANULAR SOIL.			
1	Cast iron	Open excavation	Sand	250	60	125
2	Do.	Do.	Do.	325	60	125
3	Timber construction	Do.	Do.	450	30	1,300
4	Steel construction	Pneumatic	Sand, boulders	450	68	2,700
5	Timber construction	Do.	Sand	540	75	1,700
6	Do.	Do.	Do.	650	90	1,200
7	Do.	Do.	Sand, boulders	660	101	2,100
			MIXED GRANULAR AND COHESIVE SOILS OR SILT.			
8	Steel	Open excavation	Mud, sand	450	65	1,300
9	Masonry	Pneumatic	Sand, mud	205	40	75
10	Timber construction	Do.	Silt, sand, mud	310	75	2,550
11	Cast iron	Open excavation	Gravel, clay	240	60	125
12	Do.	Do.	Sand clay	250	75	225
13	Wrought iron	Do.	Do.	285	140	1,000
14	Cast iron	Do.	Sand, clay, gravel	300	100	125
15	Steel construction	Do.	Silt, sand, clay	375	55	190
16	Cast iron	Do.	Silt, mud, clay	390	75	100
17	Steel construction	Do.	Silt, clay	450	60	700
18	Do.	Do.	Silt, clay, sand	450	60	1,200
19	Iron construction	Do.	Sand, gravel, clay	480	65	200
20	Steel construction	Pneumatic	Clay, sand	275	60	150
21	Do.	Do.	Sand, clay, gravel	350	100	1,200
22	Do.	Do.	Sand, clay, boulders	400	48	1,925
23	Timber construction	Do.	Clay, sand, gravel	400	95	4,500
24	Do.	Do.	Sand, gravel, clay	425	55	1,300
25	Timber	Do.	Silt, clay, gravel	500	75	1,800
26	Timber construction	Do.	Sand, clay	600	75	1,400
27	Do.	Do.	Sand, gravel, clay	650	80	2,000
28	Do.	Do.	Silt, sand, clay	900	45	1,700
29	Steel construction	Do.	Alluvium	370	45*	600
30	Do.	Do.	Do.	210	58	600
31	Do.	Do.	Do.	430	37*	740
			MOSTLY COHESIVE SOIL.			
32	Steel construction	Open excavation	Clay	450	75	1,500
33	Cast iron	Do.	Do.	500	60	125
34	Steel construction	Do.	Do.	700	65	1,300
35	Timber construction	Pneumatic	Do.	250	35	800
36	Brickwork	Open	Transported chalk	490	17*	675
37	Do.	Do.	Do.	520	18*	630
38	Concrete	Do.	Clay and peat	855	—	—
39	Do.	Do.	Soft blue clay	450	—	14
40	Do.	Do.	Very stiff red clay	1,912	—	10,235

* Land caisson, depth below ground.

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 the full day in the working chamber and only simple precautions are necessary to ensure their health.

For higher pressures, and to prevent compressed-air sickness, or caisson disease, the time and method of decompression become of increasing importance, while for pressures exceeding 25 to 30 lb. the working time under pressure must be reduced. *Table* XII shows typical working shift periods for the various pressures in comparison with one of the few regulations covering this subject.

TABLE XII.

MAXIMUM HOURS UNDER PRESSURE DURING ANY 24-HOUR PERIOD.

Working pressure (lb. per square inch)	Typical contractor's choice	New York State regulations*
	(Hours)	
Up to 18	8 to 10	8 hours with ½ hour (min.) rest interval
19 to 26	8	3 hours plus 3 hours after 1 hour rest
27 to 33	6	2 ,, ,, 2 ,, ,, 2 hours ,,
34 to 38	5	1½ ,, ,, 1½ ,, ,, 3 ,, ,,
39 to 43	3 to 4	1 hour ,, 1 hour ,, 4 ,, ,,
44 to 48	2	¾ ,, ,, ¾ ,, ,, 5 ,, ,,
49 to 50	1	½ ,, ,, ½ ,, ,, 6 ,, ,,

* 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.

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 are of importance.

The quantity of free air to be supplied should be calculated from the CO_2 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 $\dfrac{80}{\text{Permitted percentage increase in } CO_2}$, or for example, if the open air has a CO_2 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 1,333 cu. ft.

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 the 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.

Caisson Sickness (or "Bends").

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 Schrötter,[47] Sir Leonard Hill,[48] and Dr. F. L. Keays,[49] 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. It may be sufficient here, therefore, to deal only with the precautions that are necessary for ensuring the absolute minimum of risk to the health of the workers.

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. For this reason, exercise during decompression and massage of joints should be insisted upon.

An explanation of the cause of "bends" given by Sir Henry Japp [50] 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.

For moderate working pressures, say up to 25 lb. per square inch, exercise during decompression in combination with a 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 conse-

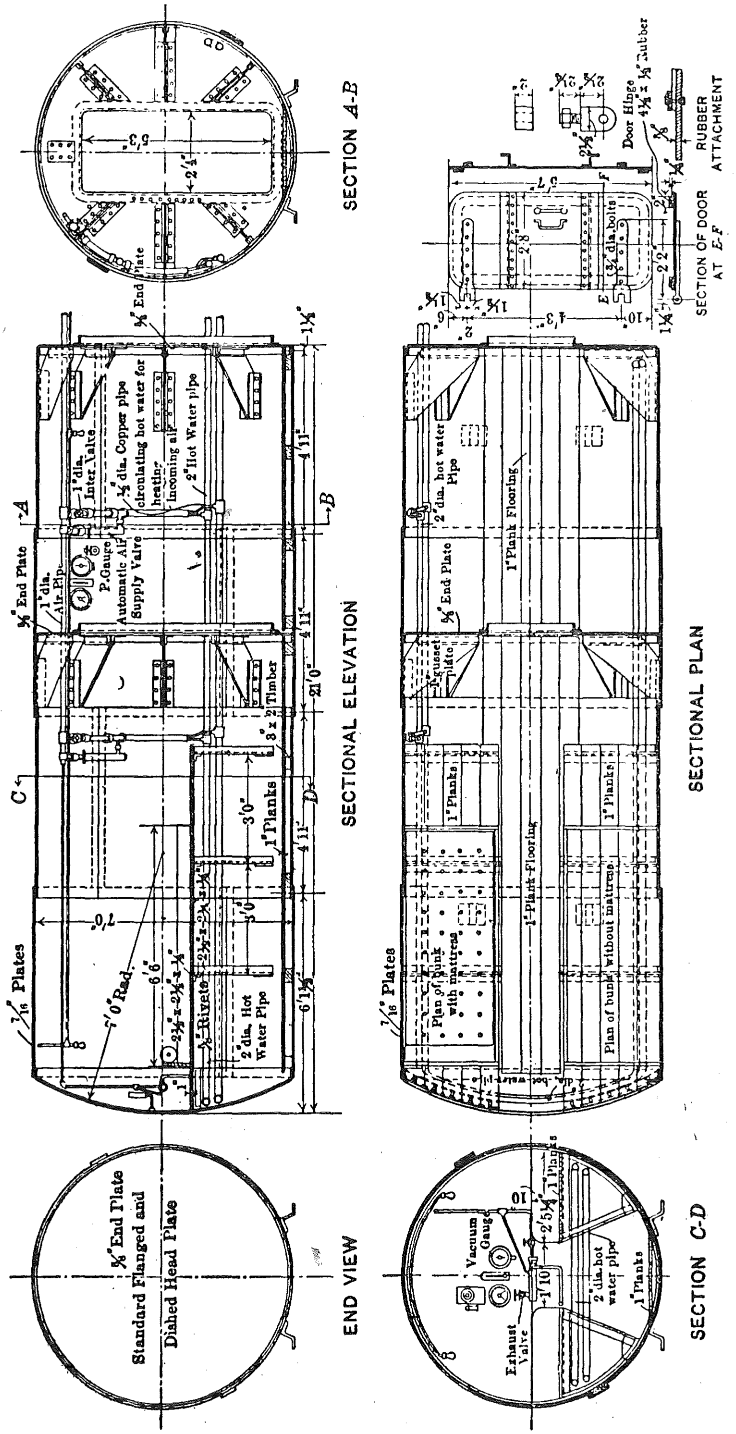


FIG. 104.—A TYPICAL RECOMPRESSION CHAMBER.

quences. 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.

For moderate and low pressures, gradual decompression in the air lock, rapidly at first and slowly as the atmospheric pressure is approached, is generally sufficient, but a recompression chamber, known as a medical lock, is provided for men to enter immediately should any symptoms of illness develop. A typical decompression chamber is shown in *Fig.* 104. Rapid recompression in this lock to about half the working pressure usually completely relieves the pains and medical attention can be given if necessary, under pressure. The pressure is then

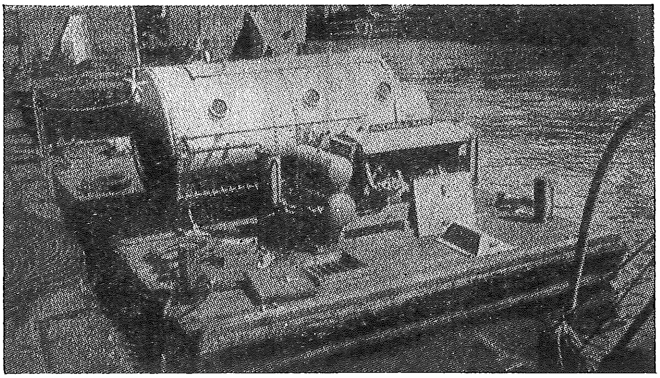


FIG. 105.—A MEDICAL LOCK.

subsequently slowly reduced as when 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.* 105 is an external view of the medical lock used by divers inspecting foundations inside the caissons for the San Francisco–Oakland Bay bridge.[52]

Various methods and rates of decompression have been used in the past, the rate of decompression in particular still differing widely. As the effects of the liberated nitrogen are to some extent dependent upon the duration of the exposure to pressure, the time of working has 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. Dr. Haldane's method, which is widely 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 $22\frac{1}{2}$ 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. Dr. Haldane has shown that there are practically no risks at all in decompression from 19 lb. gauge pressure. While the rates of decompression by some rules lead to a very long total time for decompression, Sir Henry Japp [51] used successfully a rate definitely higher than the present New York State regulations when the times of exposure are taken into account. *Table* XIII gives both rules for comparison.

TABLE XIII.

Gauge pressure (lb. per sq. in.)	Sir Henry Japp's rule		New York State regulations	
	Reduce pressure in three minutes to (lb.)	Total time in air lock after 8 hours' work (minutes)	Minutes to drop to half the gauge pressure	Total time in air lock in minutes (after hours under pressure given in brackets)
27	6	9	$2\frac{3}{4}$	18 (2)
30	$7\frac{1}{2}$	24	3	30 (2)
		After 3 hours' work		
32	$8\frac{1}{2}$	25	$3\frac{1}{3}$	32 (2)
35	10	35	$3\frac{1}{2}$	35 ($1\frac{1}{2}$)
40	$12\frac{1}{2}$	48	4	40 (1)
		After 2 hours' work		
42	$13\frac{1}{2}$	37	$4\frac{1}{3}$	42 (1)
45	15	42	$4\frac{1}{2}$	45 ($\frac{3}{4}$)
50	$17\frac{1}{2}$	48	5	50 ($\frac{1}{2}$)

It is preferable for the rapid first stage of decompression to reduce to half the absolute pressure (as in Dr. 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.

PART V

CYLINDERS AND OPEN CAISSONS: PRACTICE

Open Wells.

CHICAGO METHOD.—This method,[53] 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.

The procedure in Chicago (where the sub-soil is clay), Detroit, and other places

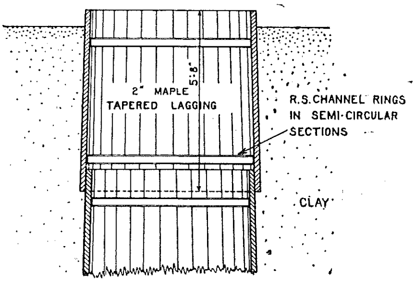


FIG. 106.—THE CHICAGO OPEN-WELL METHOD.

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.* 106 and 107 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 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-story tower of the Cleveland Union Terminal,[54] 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 belled out at the base with surfaces sloping at two in one. The foundation loads vary from

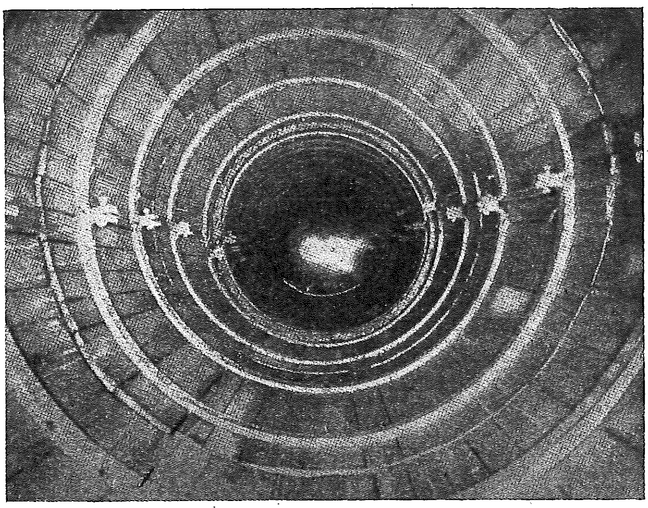


FIG. 107.—CHICAGO OPEN-WELL METHOD.

3,150 to 4,450 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 $-$ 83·0 and alternatively extending to bed-rock at an elevation of $-$ 155·0, 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.

Cylinders: General.

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.* 108), while, for the pivot piers of swing bridges, six or more cylinders are sunk around the periphery

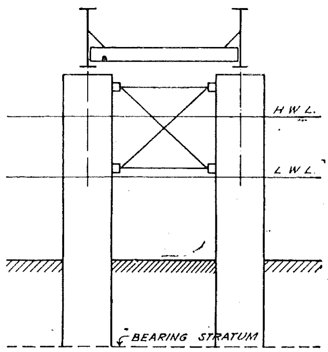


FIG. 108.—TYPICAL ARRANGEMENT OF CYLINDERS FOR BRIDGE PIERS.

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. Considerable resistance to sinking sometimes develops and kentledge 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.[55]

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 overly 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 proceeds. A well-known example of this is the Manhattan Company building[56] in New York City (*Fig.* 109). 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 141.)

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,[57] 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.* 110) were made and delivered each in one piece,

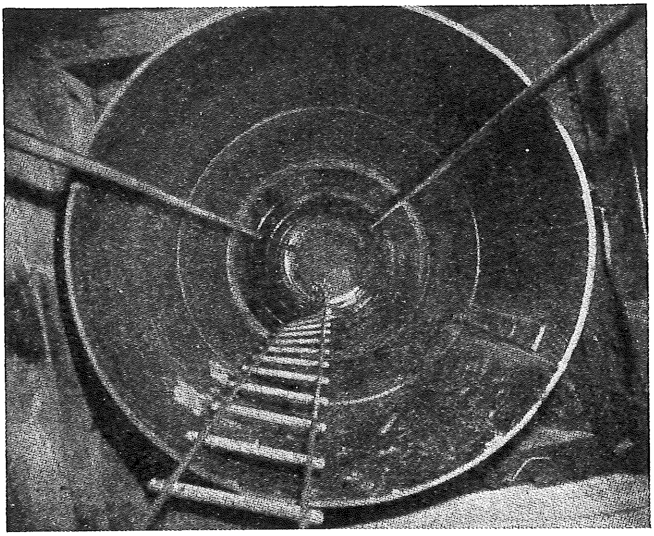


FIG. 109.—STEEL CYLINDERS USED FOR THE FOUNDATIONS OF THE MANHATTAN BUILDING, NEW YORK.

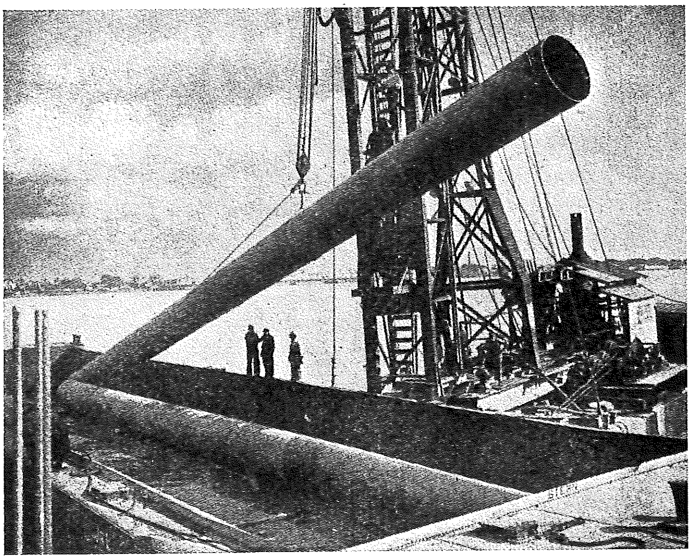


FIG. 110.—WELDED STEEL CYLINDERS UP TO 107 FT. LONG DELIVERED IN ONE PIECE FOR DRIVING BY IMPACT.

the shell being $\frac{1}{2}$ 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 1,100 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 1,000 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 $\frac{1}{2}$-in. plate. It was found that some of the hand-made 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 $1\frac{1}{2}$-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,[58] 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.

min δ =

Generally speaking, for steel cylinders a thickness of shell of less than $\frac{5}{16}$ 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 17, 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.

Sheet-piled 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.* 112 (*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.* 111), 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.* 112 (*b*). This is sometimes useful in cases of limited space or to save concrete when working to a

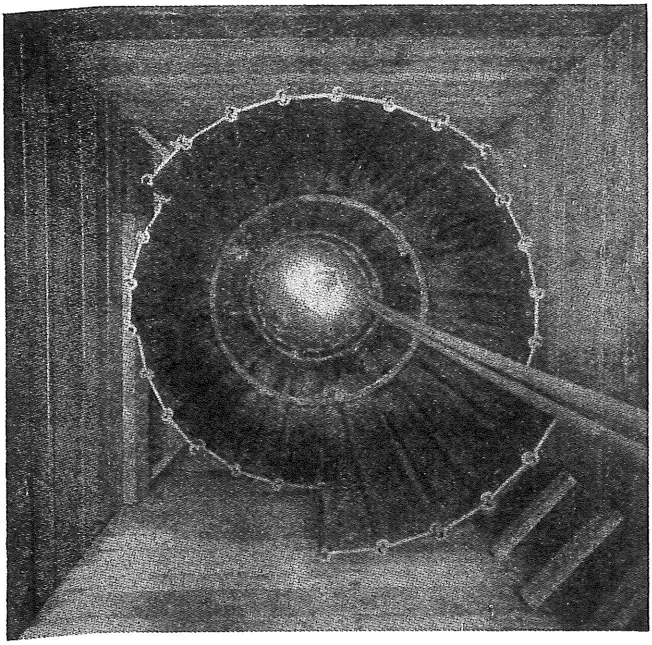


FIG. 111.—SHEET STEEL PILES DRIVEN TO FORM A CYLINDER.

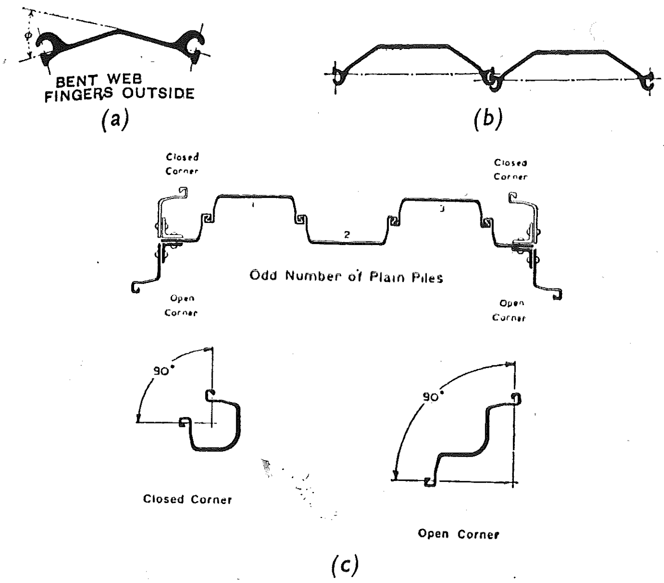


FIG. 112.

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.

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 Larssen sections as in *Fig.* 112 (*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-piled 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.* 112(*c*).

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.

Open Caisson Bridge Piers.

A circular shape in plan is usual for the smaller diameters 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.* 113, and *Fig.* 114 gives the general dimensions of the three bridges concerned.[59] *Fig.* 115 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

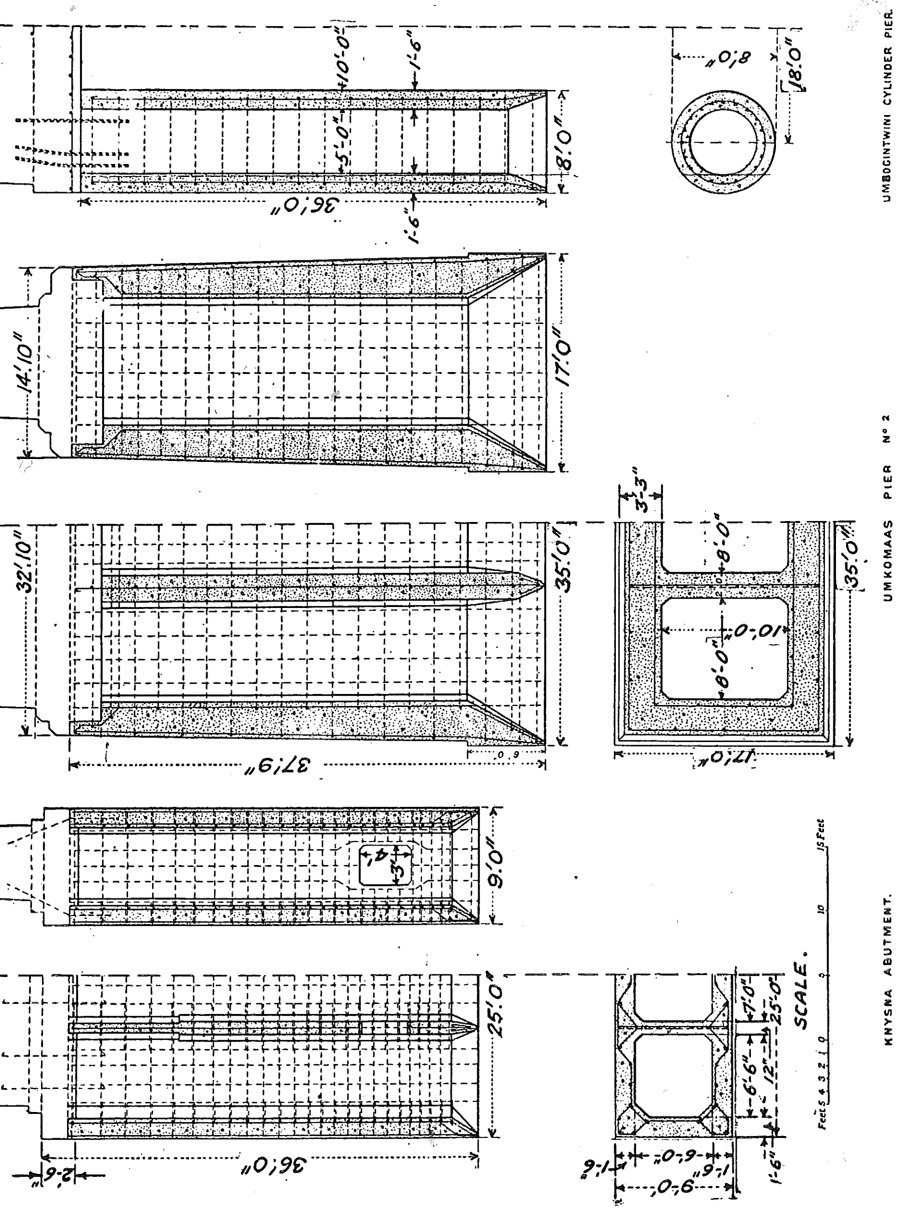


FIG. 113.—DETAILS OF OPEN CAISSONS FOR THE BRIDGE FOUNDATIONS SHOWN IN FIG. 114.

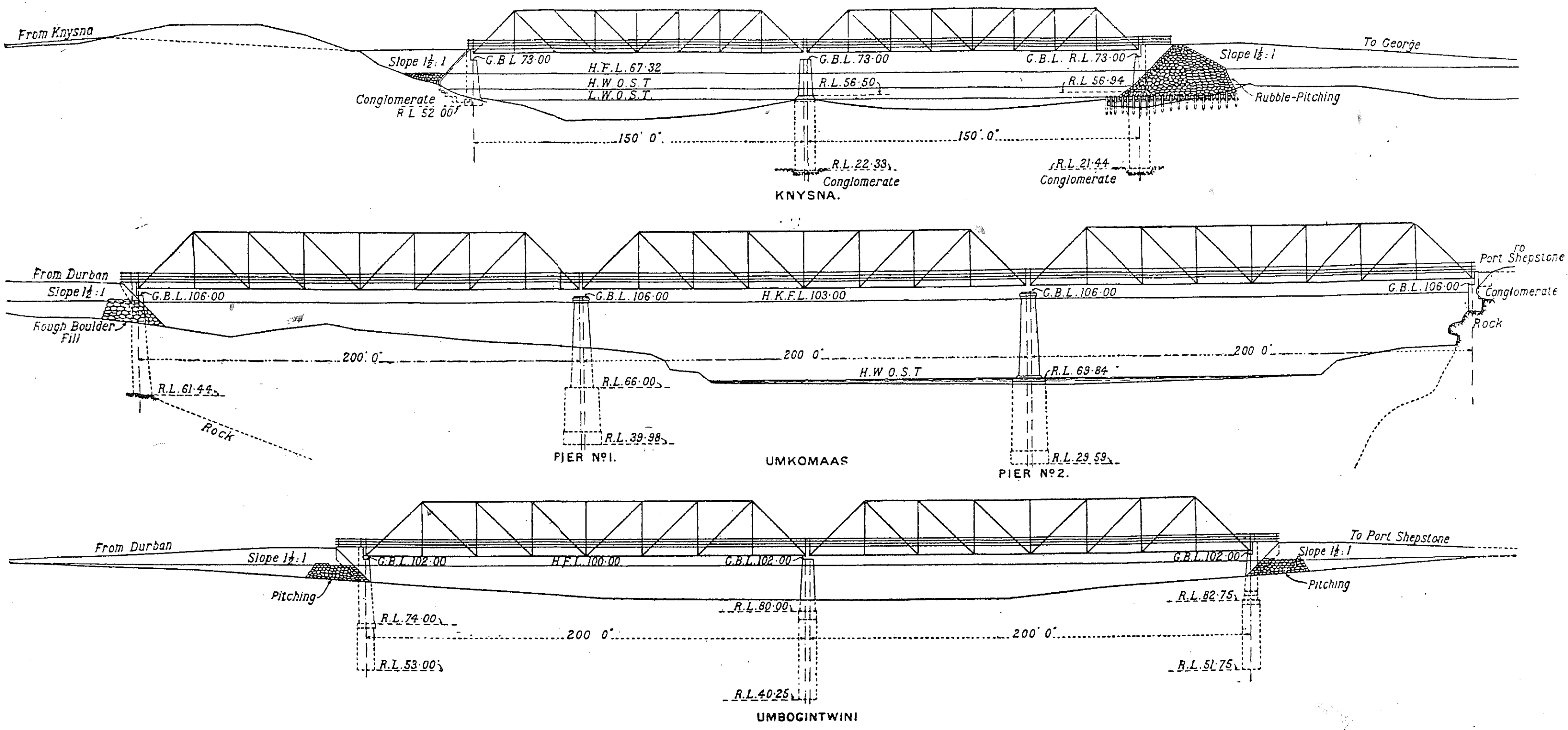


FIG. 114.—OUTLINE DIAGRAMS OF THREE BRIDGES WITH CAISSON FOUNDATIONS AS SHOWN IN FIG. 113.
(The soil penetrated is shown in *Fig.* 115.)

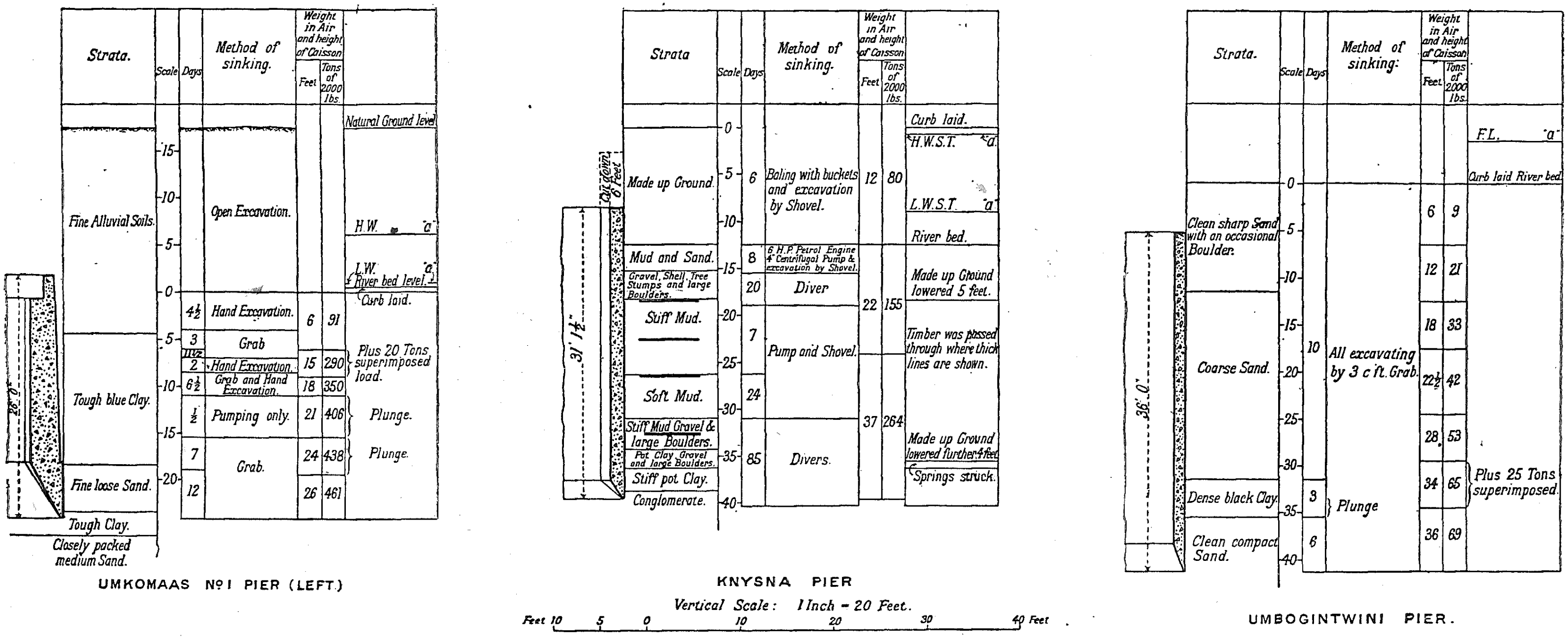


FIG. 115.—SUBSOIL PENETRATED BY CAISSONS SHOWN IN FIG. 113.

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. A mix of 1 : 2 : 4 concrete was used for the sealing layer and 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 $6\frac{1}{2}$ 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.

"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.

"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.

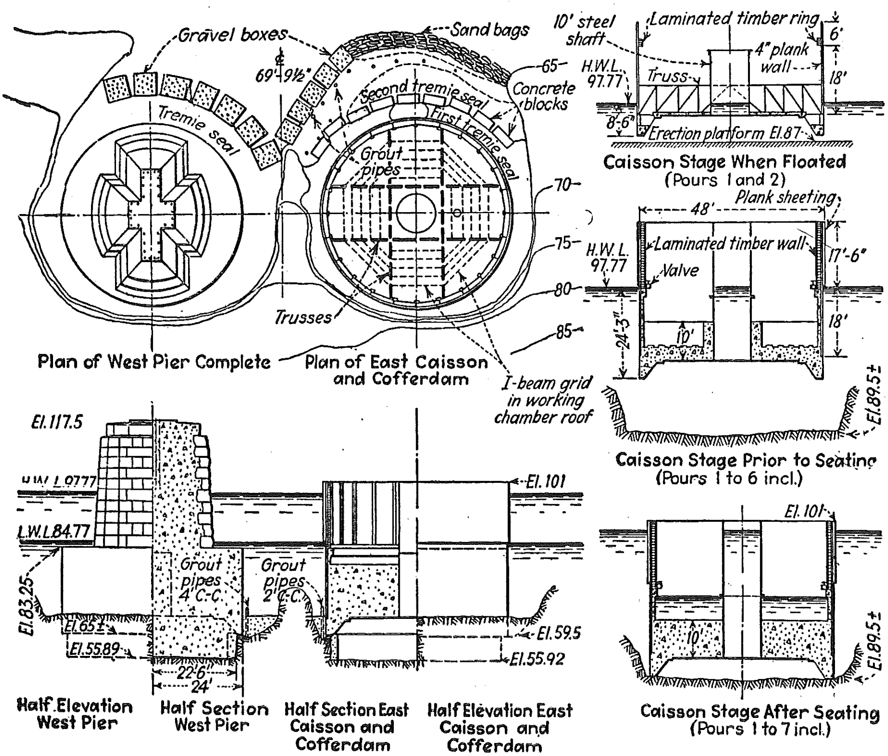


FIG. 116.—CONSTRUCTIONAL SEQUENCE FOR OPEN CAISSONS ARRANGED FOR CHANGE TO PNEUMATIC METHOD.

A recent example of this is the south pier caissons of the First Narrows (Lion's Gate) suspension bridge at Vancouver,[44] of which *Fig.* 116 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

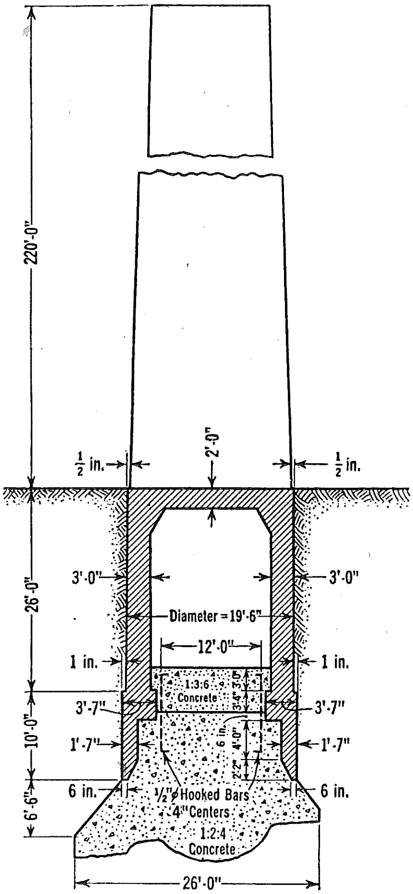


FIG. 117.—OPEN CAISSON FOR CHIMNEY WEIGHING 1,000 TONS.

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 the cofferdam method unsuitable. An example of an in-situ concrete caisson [60] for foundations to carry a load of 1,000 tons is shown in *Fig.* 117. The penetration was through layers of soft clay to fireclay at a depth of about 42 ft. Before the in-situ 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 [61] 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 2,800-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.* 118. 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 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 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.* 119. 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, so far as the writer knows, in any way with the substructure piers or main towers.

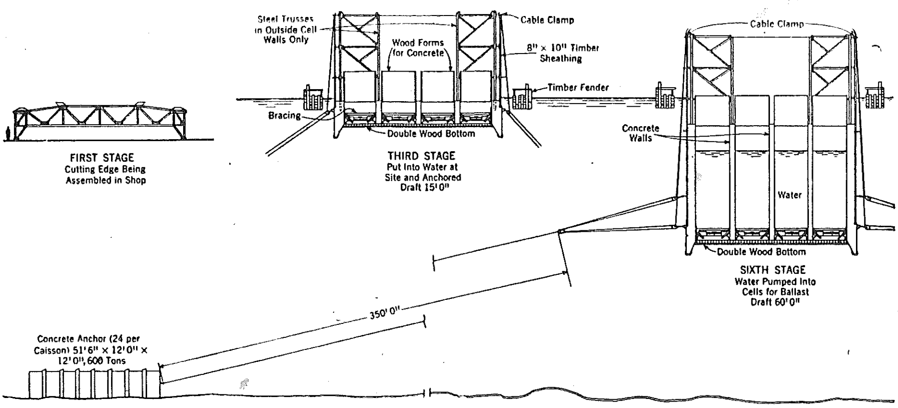


FIG. 118.—USE OF CONCRETE BLOCKS TO ANCHOR FLOATING CAISSONS.

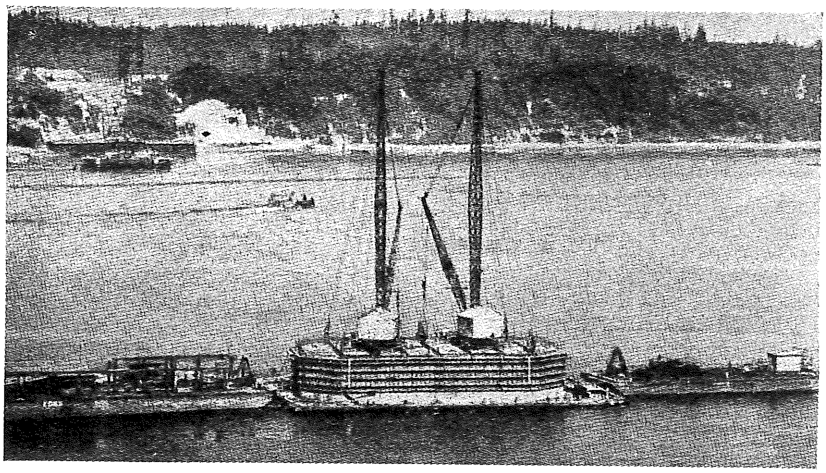


FIG. 119.—CONSTRUCTION OF PIER ON CAISSON SHOWN IN FIG. 118.

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.* 120 shows an example of this as used in the case of the Little Belt bridge [62] in Denmark, and *Fig.* 121 shows the general dimensions of the bridge. In this case the caisson was built

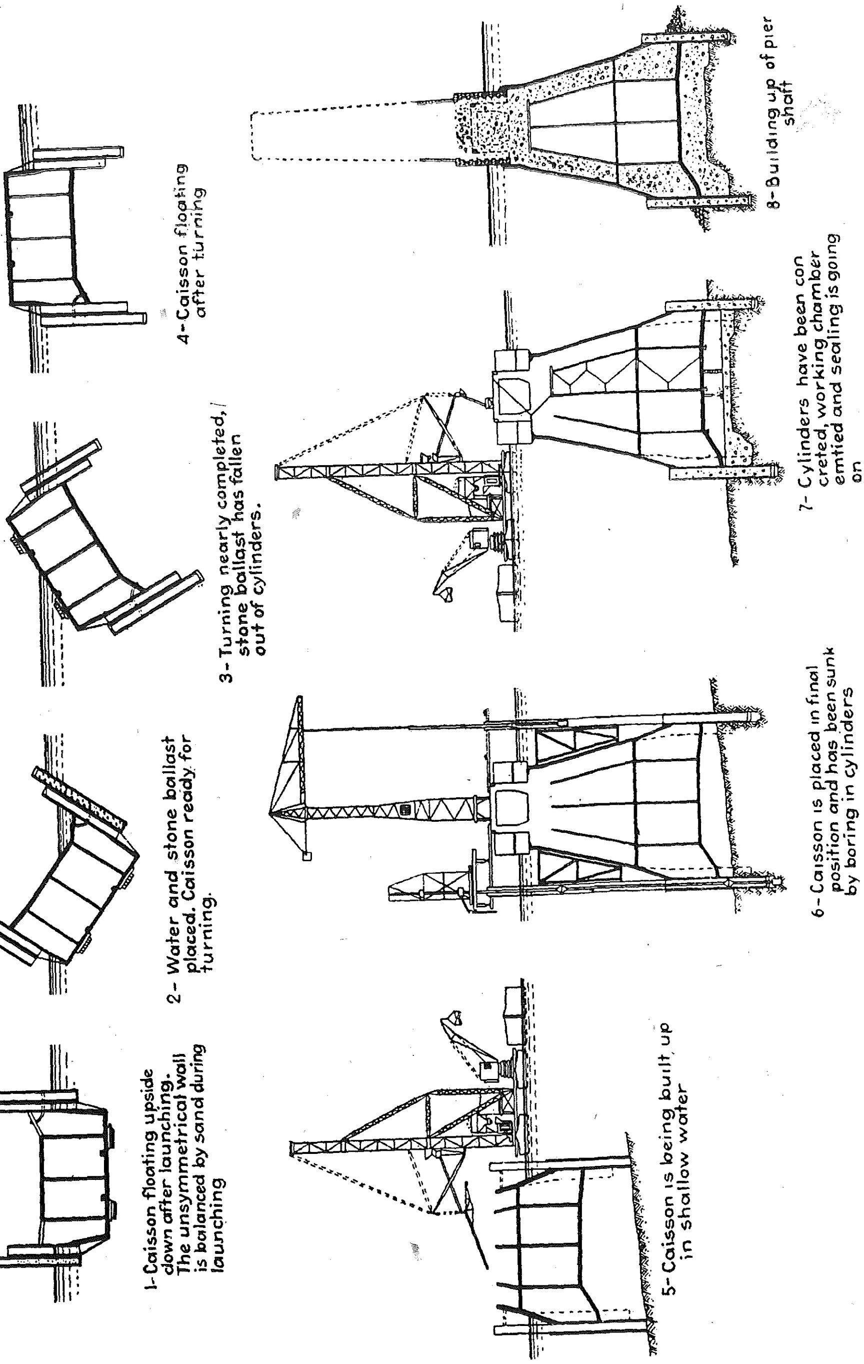


FIG. 120.—METHOD OF LAUNCHING CAISSONS IN AN INVERTED POSITION AND OVERTURNING BEFORE SINKING.

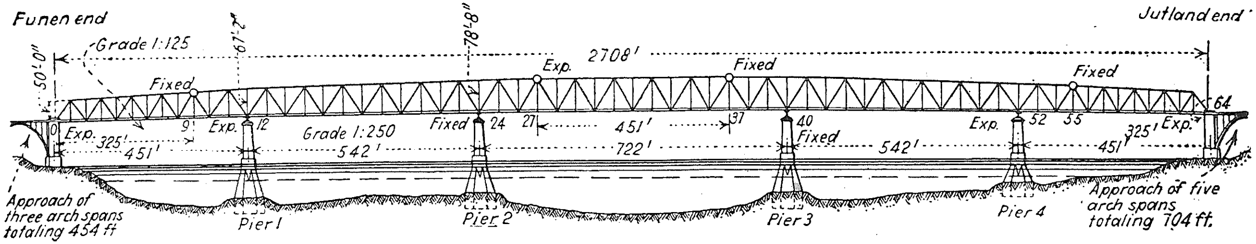


FIG. 121.—GENERAL DIMENSIONS OF BRIDGE FOR WHICH CAISSONS SHOWN IN FIG. 120 WERE USED.

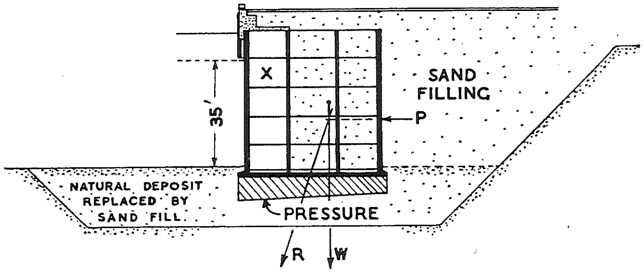


FIG. 122.—METHOD OF REPLACING AN INFERIOR SUBSOIL TO FOUND BOX CAISSONS FOR A QUAY WALL.

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 7,000 tons. The average time required for constructing a pier was reported to be as follows:

Work on caisson before launching	5 months
Finishing caisson and placing it	6 ,,
Making caisson ready for boring	3½ ,,
Boring and sealing in pipes	2 ,,
Work in chamber	1½ ,,
Finishing pier and pier shaft	6 ,,
Total	24 ,,

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.* 122) 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 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

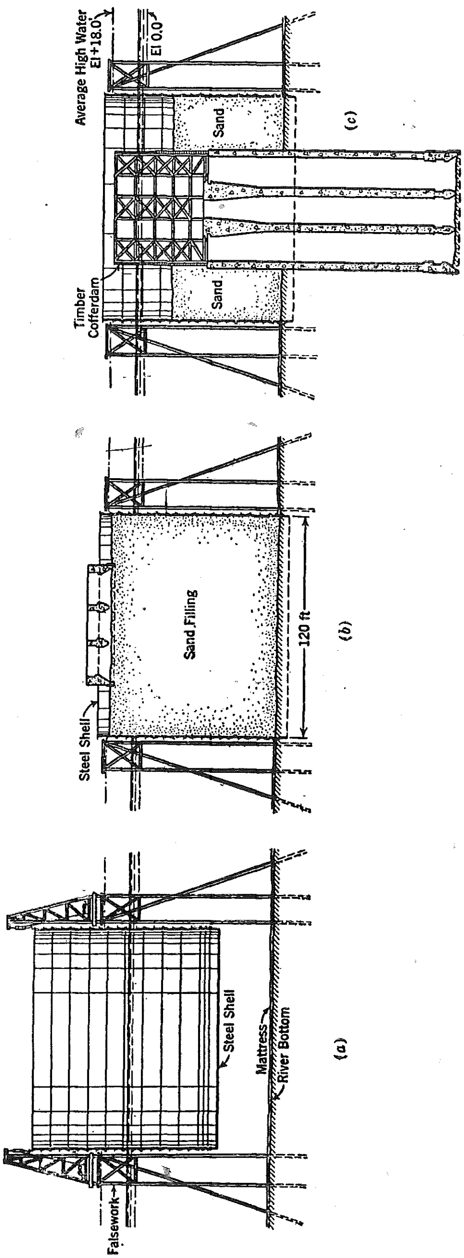


FIG. 123.—THE "SAND ISLAND" METHOD OF SINKING CAISSONS.

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,

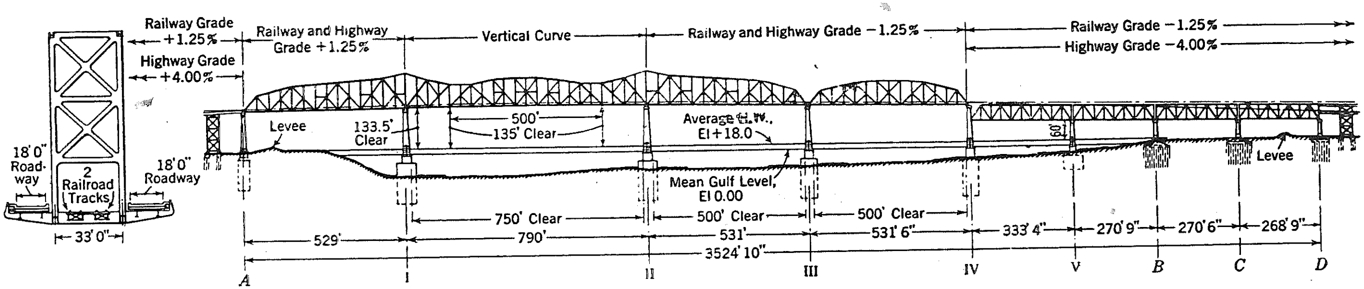


FIG. 124.—GENERAL DIMENSIONS OF BRIDGE FOR THE FOUNDATIONS OF WHICH THE SAND-ISLAND METHOD SHOWN IN FIG. 123 WAS USED.

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. In the case of the subsoil or the river conditions making the stability of caissons difficult, the sand island method may become suitable. The method consists essentially of sinking 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.[63] 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 dredging through open caissons to depths which would have been impossible by the use of pneumatic caissons. The method used is shown in *Fig.* 123.

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.* 125) 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 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

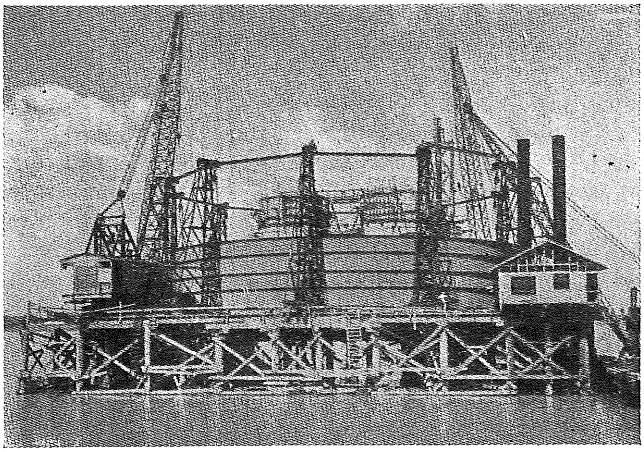


FIG. 125.—SINKING THE STEEL SHELL FOR THE "SAND-ISLAND" METHOD.

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.

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 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.* 126) each 29 in. internal diameter, for an extension of an electricity generating station.[64] The tubes were driven by a 90-ft. pile frame, using a single-acting steam hammer having a 7,500-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 shown in *Fig.* 127 (*a*). The loads supported were as great as 1,356 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.* 127 (*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

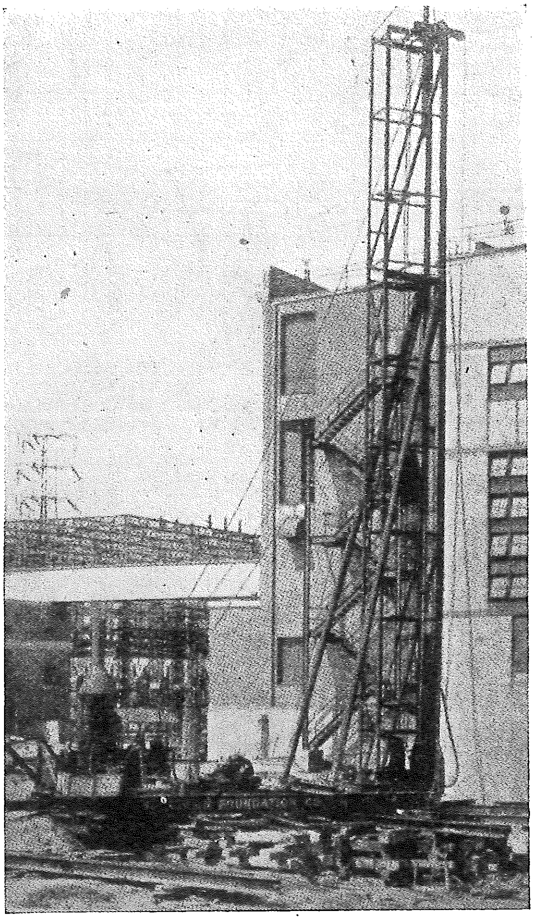
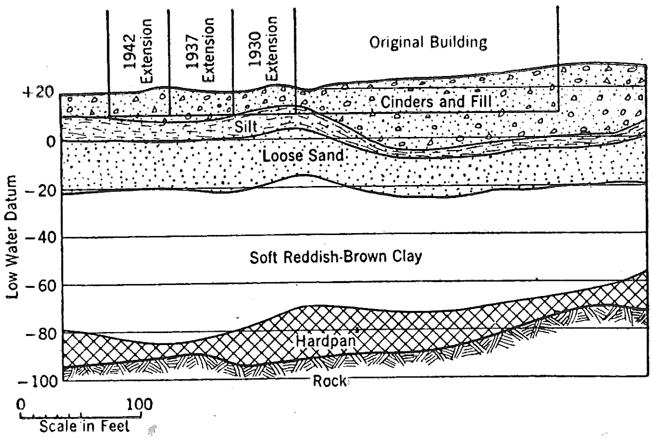
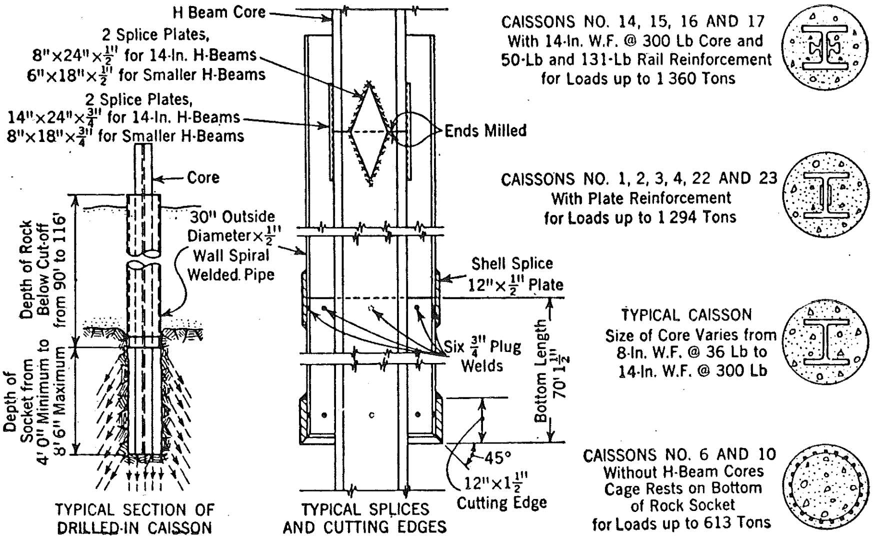


FIG. 126.—DRIVING 30-IN. DIAMETER TUBES 100 FT. TO 116 FT. TO ROCK.

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 4,000-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



(a)



(b)

FIG. 127.—METHOD OF SOCKETING ROCK TO TAKE CONCENTRATED LOADS AND OCCASIONAL UPLIFT.

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

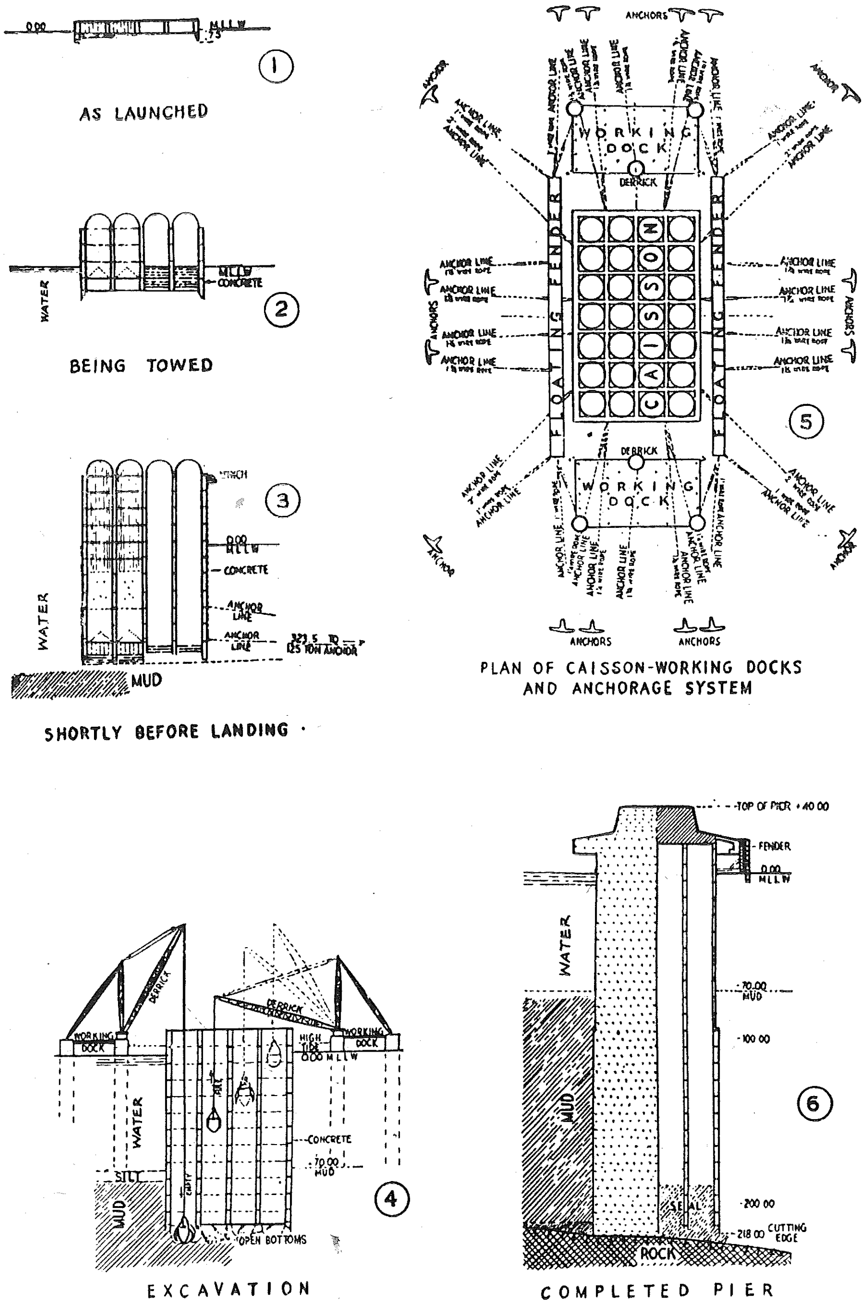


FIG. 128.—METHOD OF SINKING CAISSONS FOR THE WEST CHANNEL PIERS OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE.

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.* 118.

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 (65) 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.* 128 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.* 130) 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 101 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.* 129), control of the air pressure in groups of cells enables the caisson to be more easily kept vertical.

The following extract (66) describes the construction and sinking of the caisson (W4), the largest ever constructed, and which is typical of the other three.

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 $\frac{5}{16}$-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

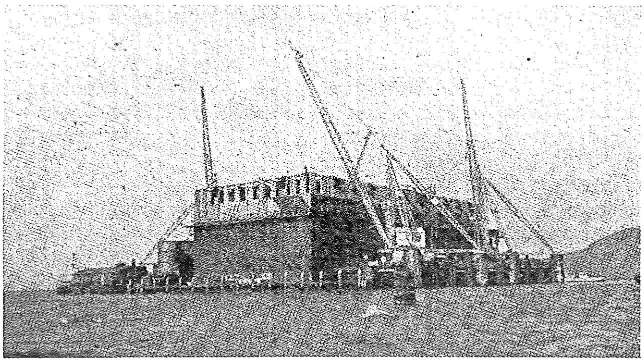


FIG. 129.—CAISSON BEING SUNK FOR A PIER OF SAN FRANCISCO-OAKLAND BAY BRIDGE.

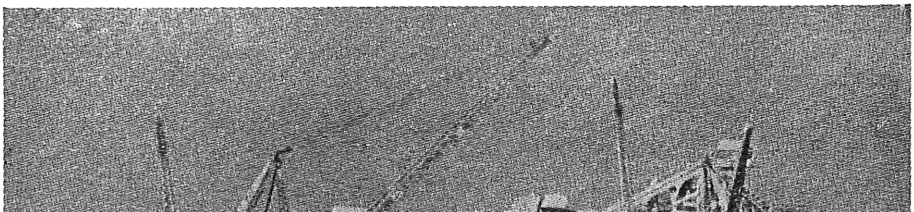


FIG. 130.—CAISSON FOR CENTRAL ANCHOR PIER, SAN FRANCISCO-OAKLAND BAY BRIDGE.

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 30,700 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.* 130]. 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."

PART VI

PNEUMATIC CAISSONS: PRACTICE

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.

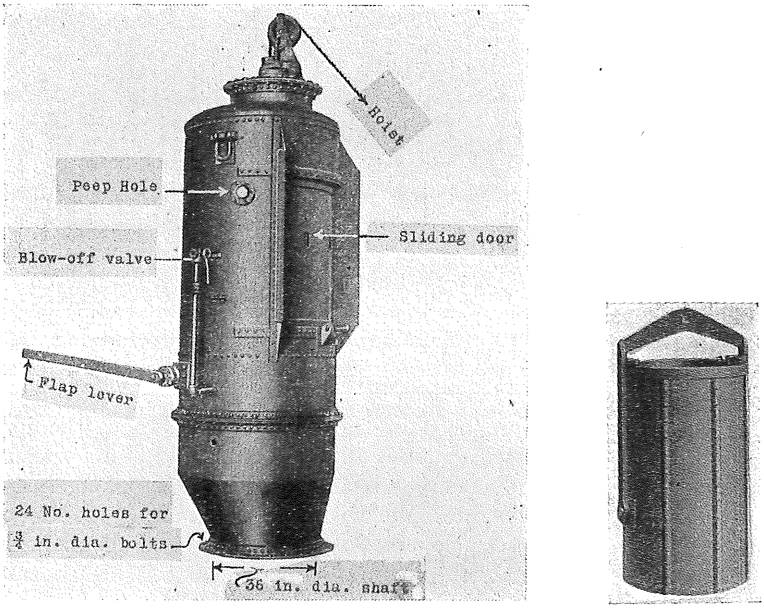


FIG. 131 (*a*) AND (*b*).—SLIDING-DOOR TYPE MATERIAL AIR LOCK, AND SKIP (OR BUCKET).

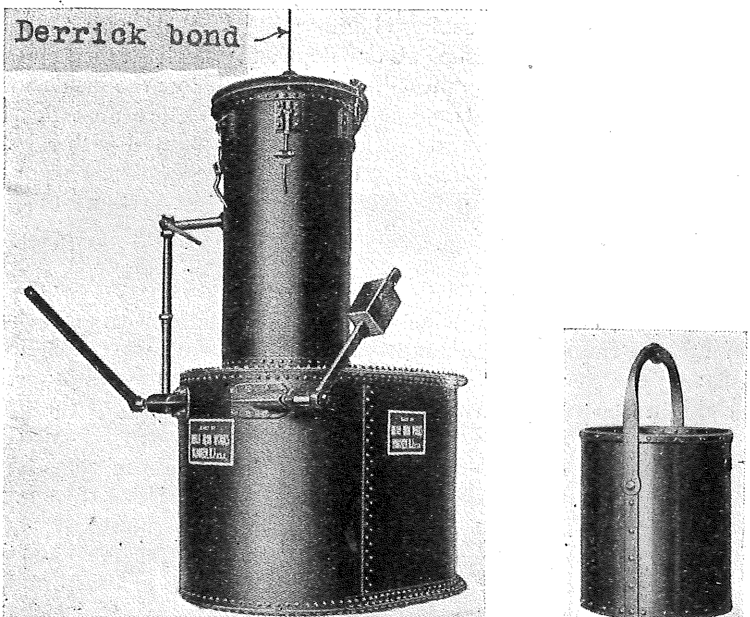


FIG. 132 (*a*) AND (*b*).—POT-LID TYPE COMBINED MATERIAL AND MAN LOCK, AND SKIP (OR BUCKET).

A typical material lock of the sliding side door type is shown in *Fig.* 131, while a combination material and man lock of the pot-lid type is shown in *Fig.* 132. 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

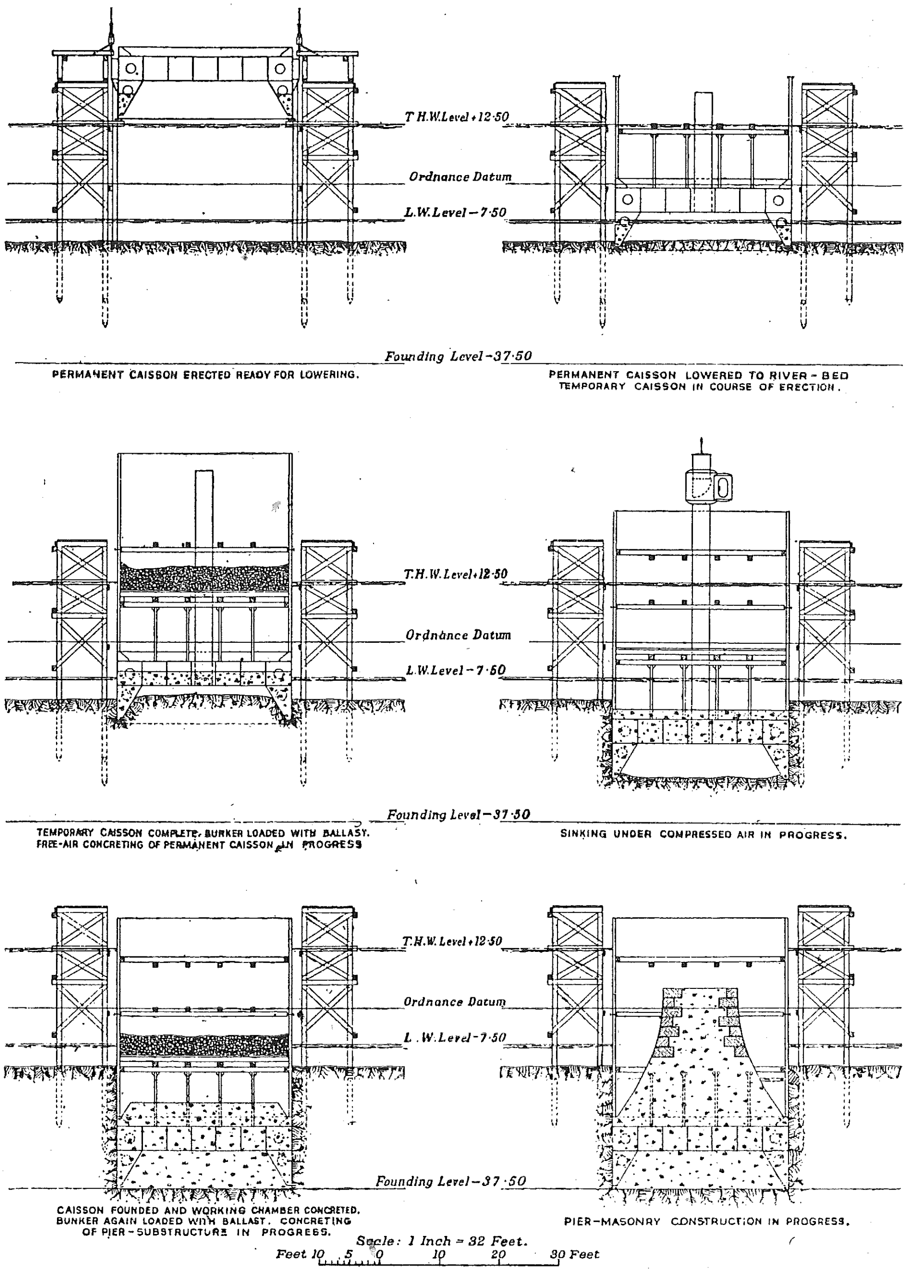


FIG. 133.—STAGES IN THE SINKING OF CAISSON FOR RIVER PIER OF LAMBETH BRIDGE, LONDON.

[Details of the caissons are shown in *Fig.* 134.]

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

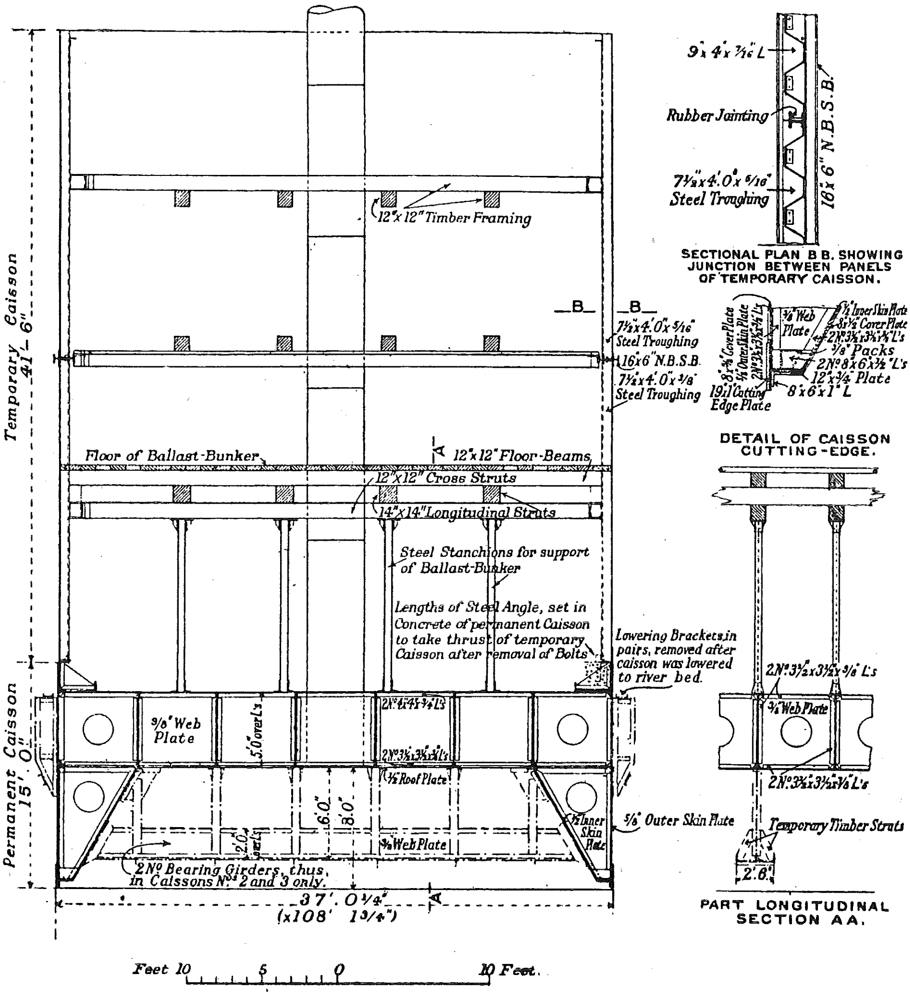


FIG. 134.—DETAILS OF CAISSON SHOWN IN FIG. 133.

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.

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.* 133 and the details of the caissons in *Fig.* 134. 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 (68) relating to the sinking of the caissons.

"Each of the four piers is founded on a single steel caisson [*Fig.* 134], which is a rectangular steel structure measuring 108 ft. $1\frac{3}{4}$ in. by 37 ft. $\frac{1}{4}$ 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 $\frac{5}{8}$-in. plate and the sides and roof of the working chamber of $\frac{1}{2}$-in. plate. Twenty plate girders, measuring 5 ft. over angles and set 5 ft. $1\frac{1}{2}$ 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 by 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 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 ½-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.* 134] 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 7½-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:

	Tons
Dead weight of permanent and temporary caissons, concrete, shafts, locks, and timber	3,100
Average displacement of above. (This varied to some extent with the tidal height and with the depth of the caisson in the river-bed)	1,300
Balance	1,800
Displacement of working chamber	720
Balance	1,080
Deduction for upward force of 2 lb. per square inch of air pressure above hydrostatic	510
' Sinking effort ' under working conditions	570

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,

	Tons
' Sinking effort ' under working conditions, as above	570
Add downward load resulting from reduction of air pressure by about 6 lb. per square inch	1,520
' Sinking effort ' while ' blowing down '	2,090

Under the action of this force of approximately 2,000 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.* 135 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[69] 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.* 135]. 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. ong and 5 ft. $2\frac{1}{2}$ in. deep. The strakes were formed of $\frac{1}{2}$-in. steel plates with 5-in. by 3-in. by $\frac{1}{2}$-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. ½ in., of which 26 ft. 6½ 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 1¼-in. bolts each 16 ft. long, which fitted into nuts left permanently embedded in the concrete. The function of these bolts was to minimise 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.* 135, 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. Building of the concrete substructure and sinking of the caisson were carried out simultaneously.

"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 minimise 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 minimise 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

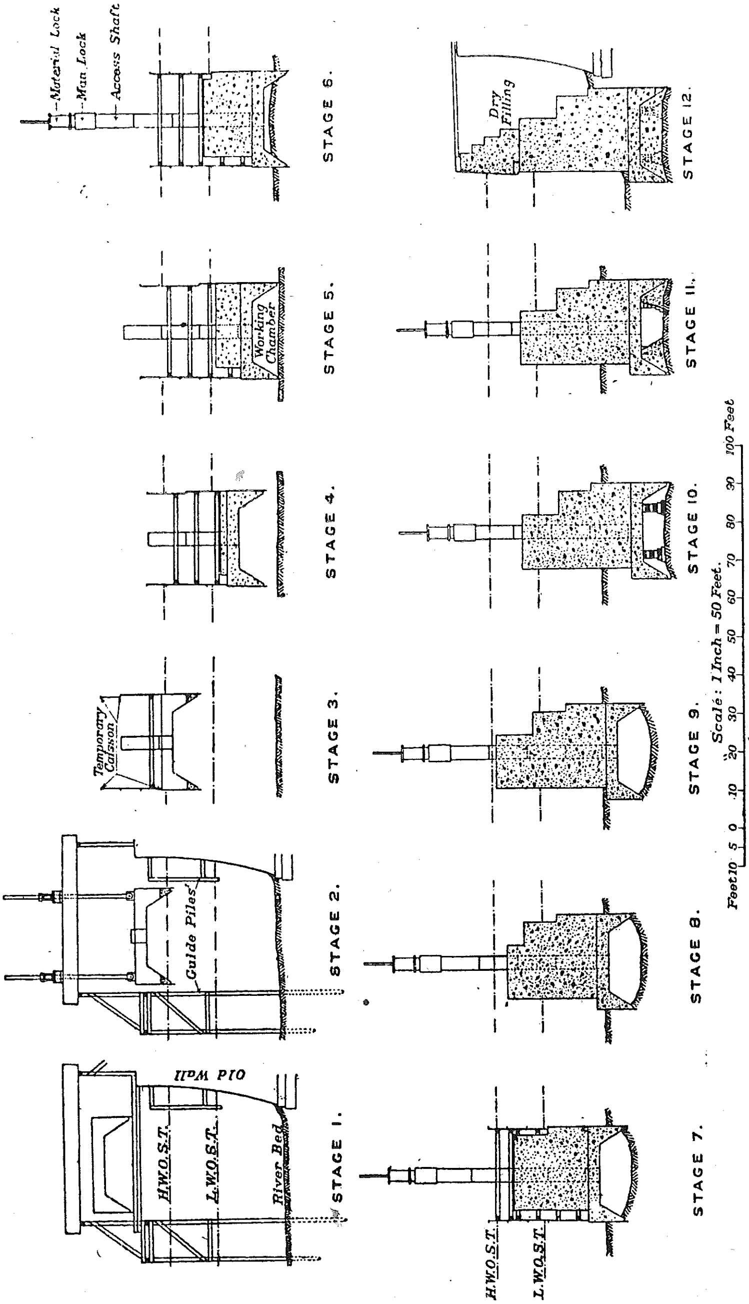


FIG. 135.—SUCCESSIVE STAGES IN SINKING PNEUMATIC CAISSON FROM A STAGING.

in under five days, thus allowing the strakes 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 1 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 1 in 34 at the time of grounding, but gradually increased and when excavation was started it had reached a value of 1 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 1 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 canted 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 canted 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 $1\frac{1}{2}$ 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 $\frac{1}{2}$ 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 $\frac{1}{2}$ 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. 136 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

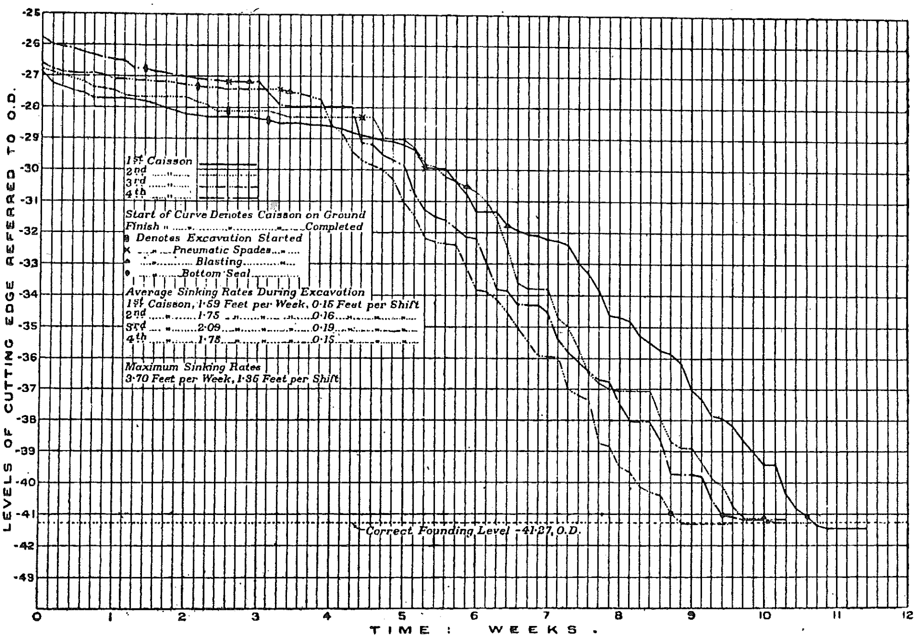


FIG. 136.—RATE OF SINKING CAISSON SHOWN IN FIG. 135.

that each skip held $\frac{1}{4}$ 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."

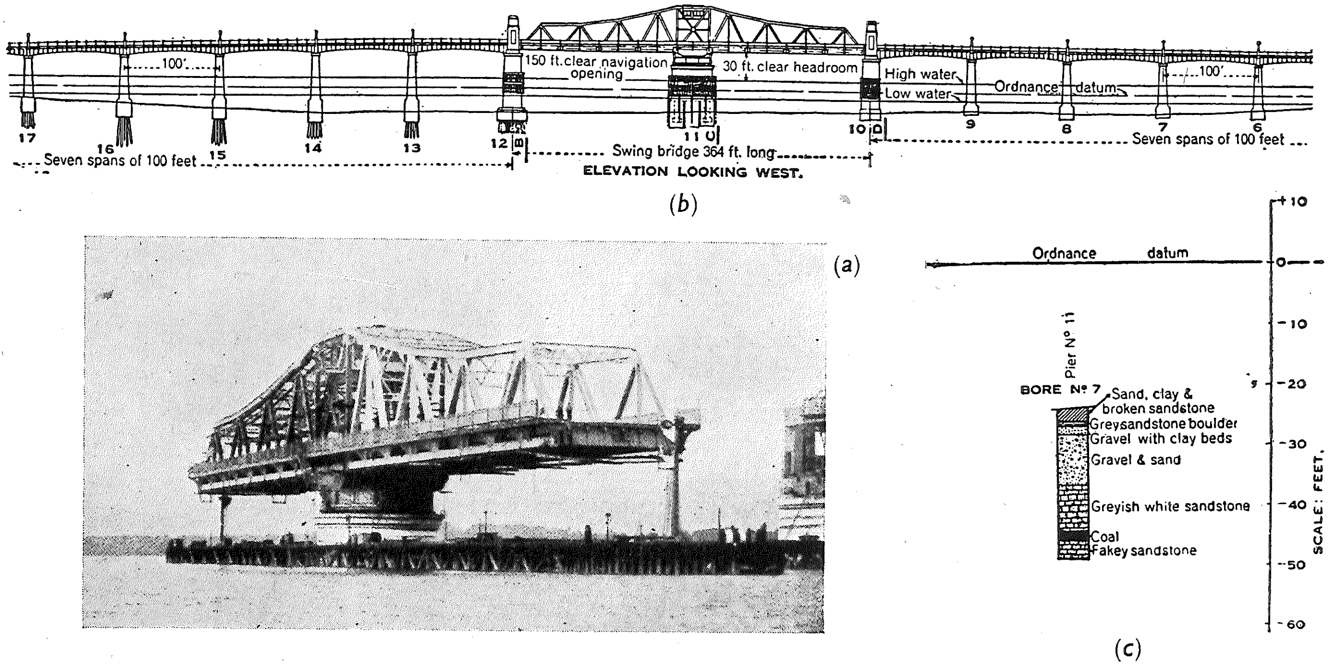


FIG. 137.—SWING SPAN OF BRIDGE ON PNEUMATIC CYLINDER CAISSONS SHOWN IN FIG. 138.

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.* 137 (*a*) to (*c*) and *Fig.* 138; the arrangement of the access through the air locks is shown in *Fig.* 139. The total load of the moving span is 1,600 tons,

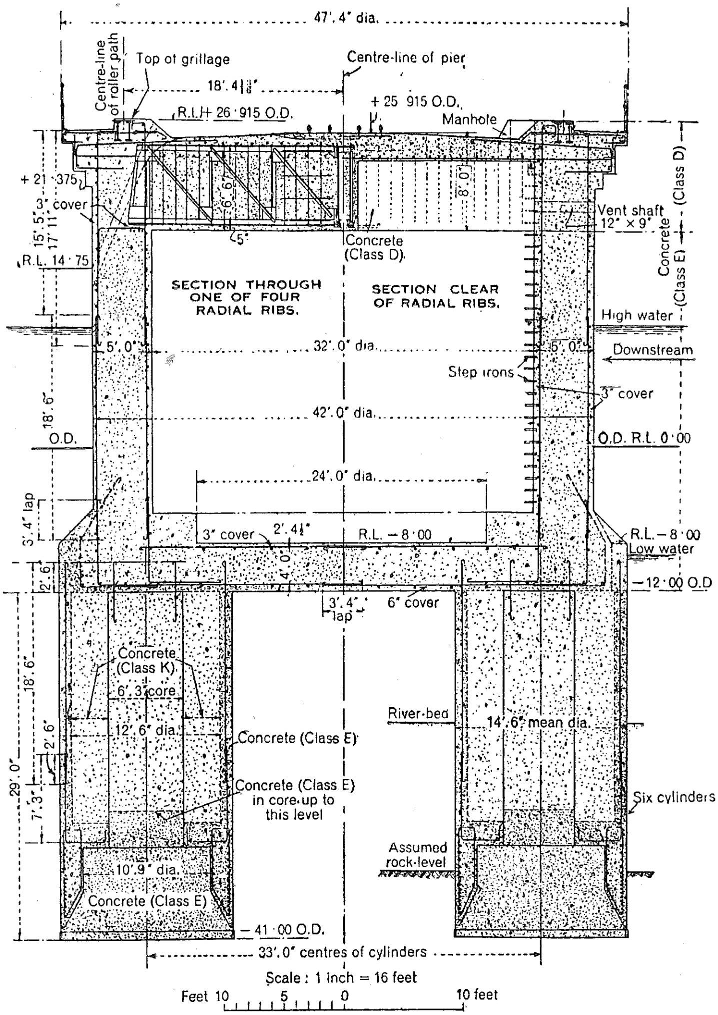


FIG. 138.—DETAILS OF PNEUMATIC CYLINDER CAISSONS FOR SWING SPAN SHOWN IN FIG. 137.

and the total load on the six cylinders including the piers themselves is 4,200 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 de-watering, 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

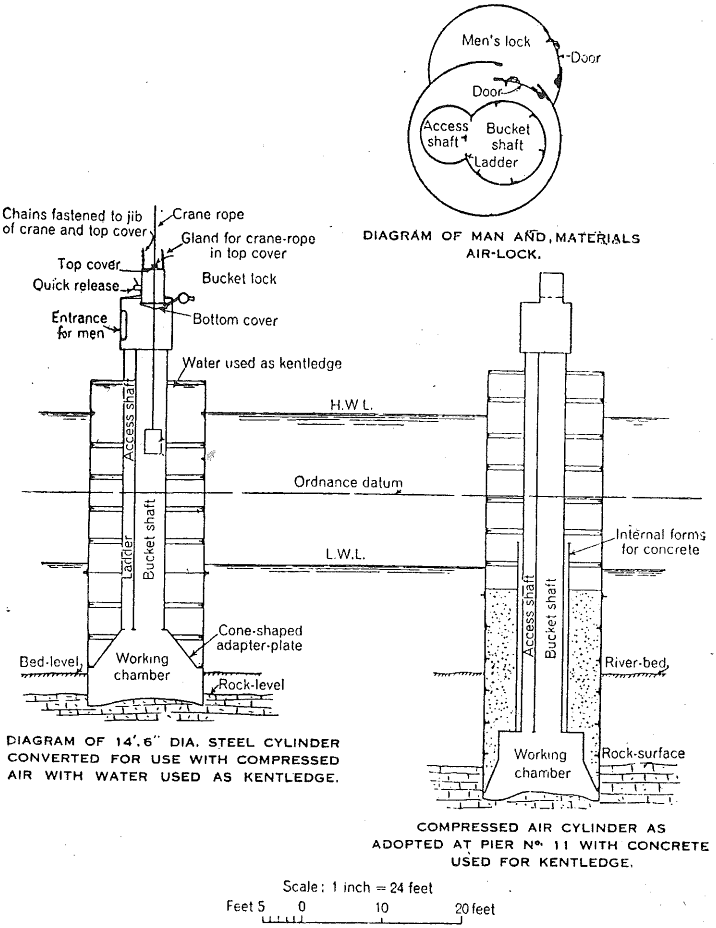


FIG. 139.—TYPICAL EXAMPLES OF AIR LOCKS FOR PNEUMATIC CAISSONS.

foundations of the swing span is from a paper by Mr. Guthrie Brown [67] 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.

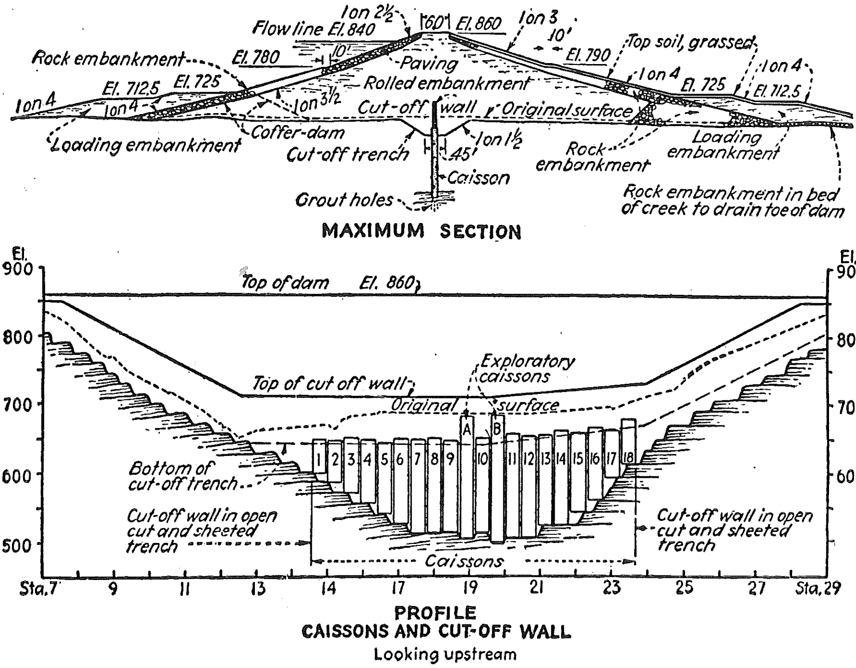


FIG. 140.—CAISSONS FOR CUT-OFF WALL OF DAM.

The base of the slab is 3 ft. below low 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

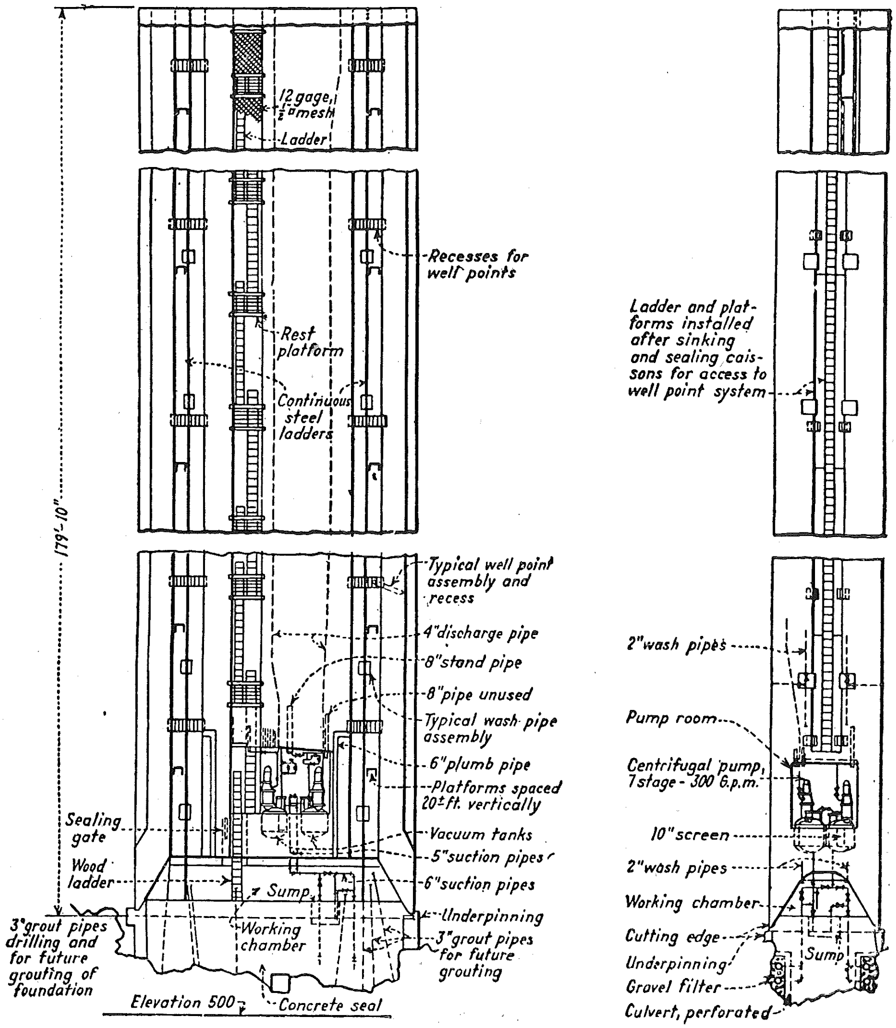


FIG. 141.—DETAILS OF PNEUMATIC CAISSONS FOR CUT-OFF WALL SHOWN IN FIG. 140.

dam[70] of the Delaware water supply project, of which the caisson cut-off wall is seen in *Fig.* 140, 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.* 141 will be seen the sleeves for four wellpoints 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.

PART VII

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 " Phœnix "

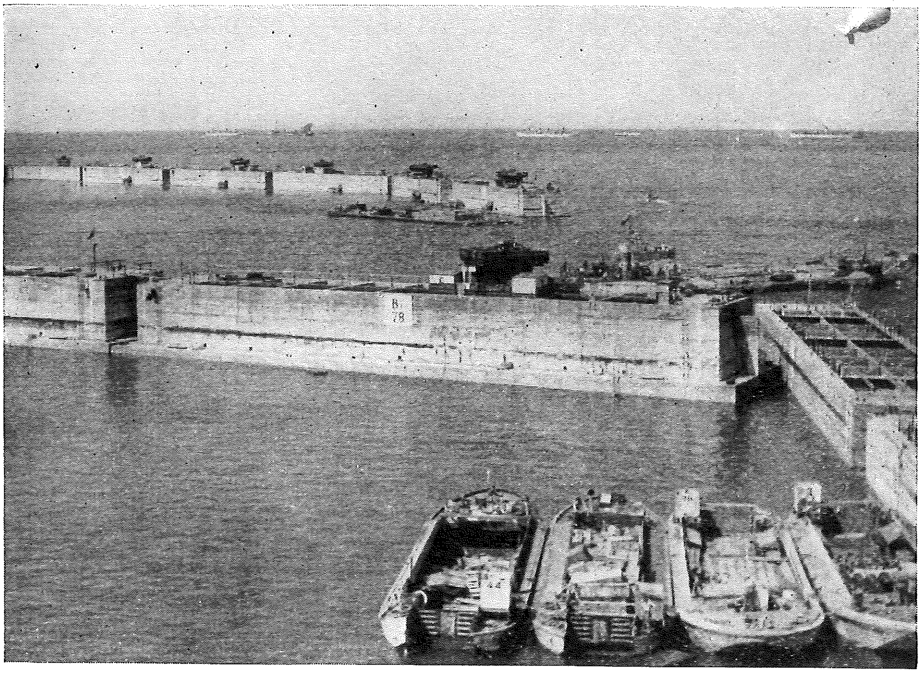


Fig. 142.—Box Caissons at " Mulberry " Harbour.

reinforced concrete box caissons (*Fig.* 142) 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. Descriptions of the harbours have already 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., 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.* 143.

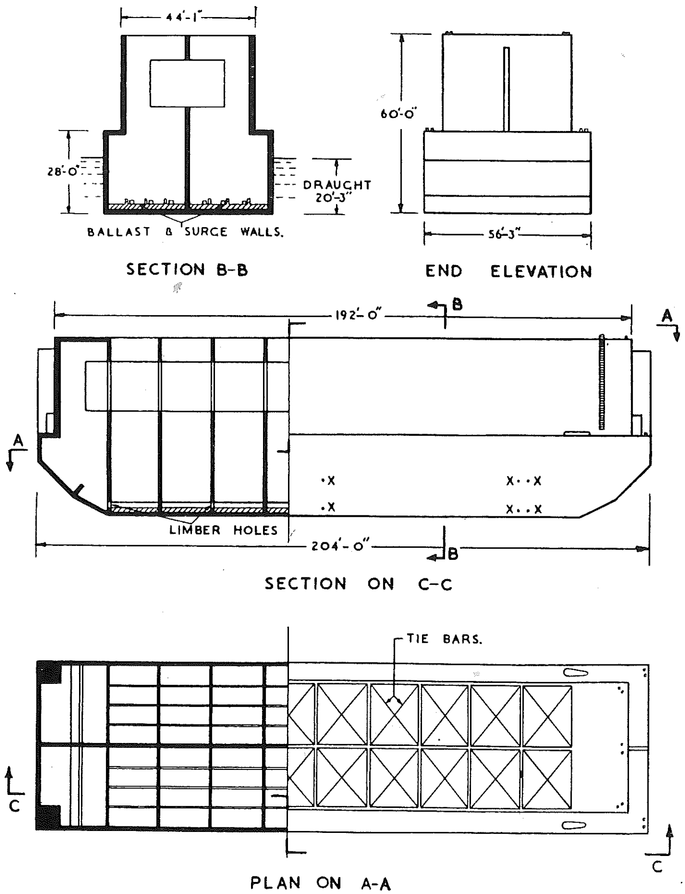


FIG. 143.—BOX CAISSONS AT "MULBERRY" HARBOUR.

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 de-watered, the dock being refilled when the units were practically complete.

METHOD OF TOWING.—Towing was generally by means of a towing bridle of 4-in. wire cable with spring shock absorbers (*Fig.* 144), 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.

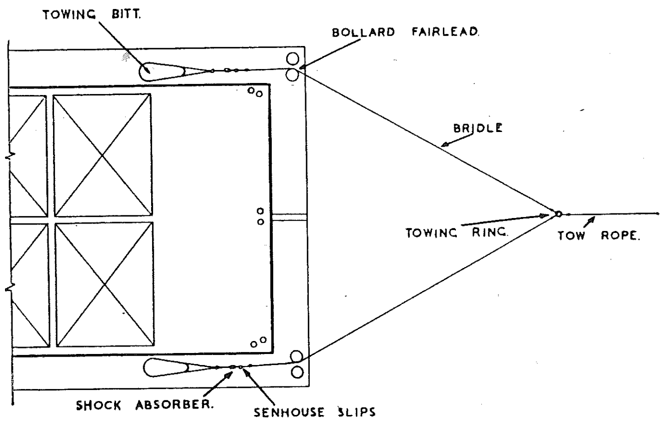


FIG. 144.—METHOD OF TOWING BOX CAISSONS FOR "MULBERRY" HARBOUR.

SINKING PROCEDURE.—Generally, sinking was effected by opening the valves (*X*) in *Fig.* 143 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.

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Fig. 95, Mr. A. G. Vaughan-Lee, M.Inst.C.E.
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