*Discussion on Paper No. 6588**

Cofferdams

by

Savile Packshaw, B.Sc., M.I.C.E.

Mr H. J. B. Harding (Consulting Engineer) said that one of the things to watch for in cofferdams was a blow in the bottom. These were largely caused by people ignoring Mr Packshaw's advice on the folly of unwise economy, and by not allowing enough penetration of their piles; very often by not knowing what the ground was, and also by not relieving water pressure. A wise engineer making a cofferdam, especially in cobbles or gravel, should contemplate the possibility of piles coming out of their clutches.

72. At the end of his Paper Mr Packshaw had some comments on concreting under water, and that was a method which could be used far more often than was customary. People were frightened of doing it, but in a properly conducted job it was easy, and it saved a great deal of risk. Engineers were often frightened of underwater concreting, and then they regretted it at leisure when they had an expensive blow in the bottom. It was far wiser to prepare for an eventuality rather than try to deal with it after it had occurred.

73. One of the developments in cofferdams, apart from steel piling, was the use of steel walings. Mr Storey Wilson had pioneered using designed structural frames and also in the use of concrete frames built on the ground at each level of excavation. By using a heavier pile and keeping frames well apart it was sometimes possible to use mechanical excavation between the walings.

Mr V. H. Collingridge (Director, Mowlem (Civil Engineering) Ltd) said the Author had stated in § 2 that there was no hidden factor of safety in cofferdam construction; there were indeed many potential hazards, not the least of which was that heavy loads were constantly being swung over cofferdam bracing, and a large framed excavation was very vulnerable to an accidental crane failure. At Rathbone Place[35] in 1956 the top frame was designed to carry a vertical load of 2 tons applied at any point and the frames were supported by bracketed king-piles at alternate intersections. King-piles also contributed very substantially to the ease of erection of the framing.

75. It was not always appreciated that "the very large forces derived from water pressure" might act, not upon the side supports of the cofferdam, but on the ground below it.

76. In artesian water-table conditions, the water itself might be never seen in the excavation, but its head must be relieved to maintain stability in the bottom of the excavation; if pumping were stopped even for a matter of minutes, natural hydrostatic head was restored. The consequences could be safeguarded under certain conditions by relief wells, but if conditions were not suitable for this remedy, pumped wells, spare pumps and stand-by power must be provided. A case in point was the cooling-water pumphouse at the Shellhaven Refinery.

77. This cofferdam was bottomed up at 30 ft depth in soft clay, sheet piles were driven 60 ft to toe 10 ft into the underlying gravel.

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* *Proc. Instn civ. Engrs*, vol. 21, February 1962, pp. 367–398

[35] The references are given on p. 124.

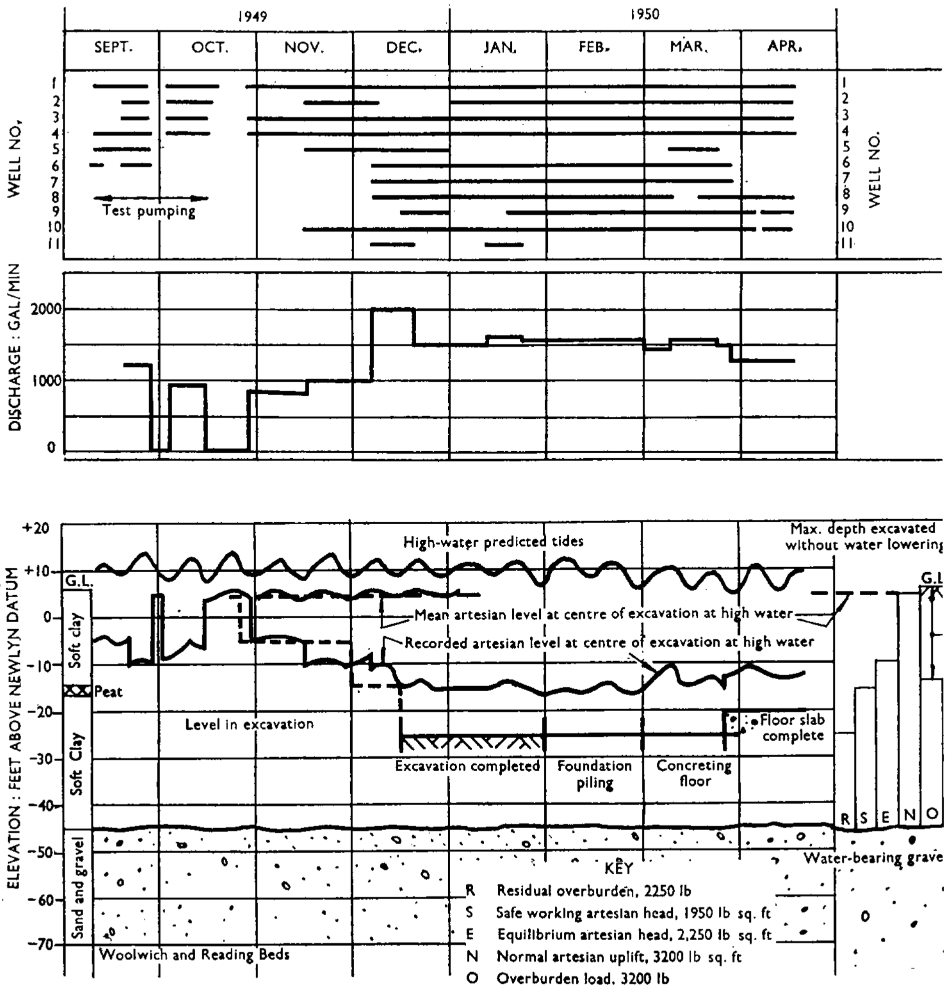


FIG. 33: COOLING-WATER PUMPHOUSE EXCAVATION, SHELLHAVEN, ESSEX. PUMPING RECORD

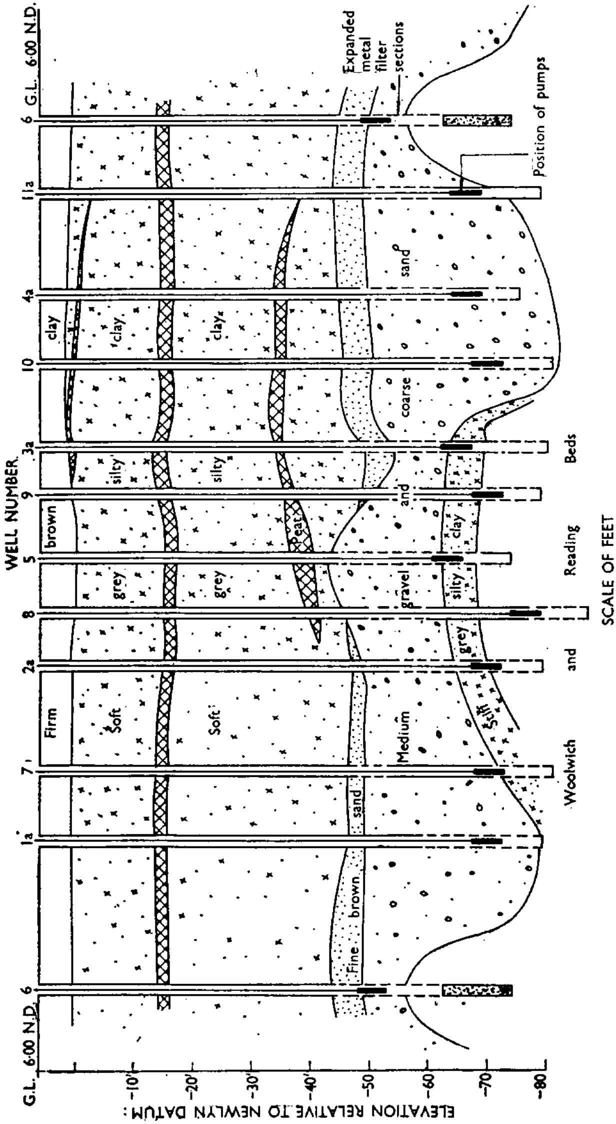


FIG. 34: DEEP WELLS AT SHELLHAVEN, ESSEX. SECTION OF STRATA SHOWING POSITIONS OF PUMPS AND FILTERS

78. There was much discussion on whether the piles should not finish a few feet below formation in impermeable clay. Had pumping failed for some reason, in spite of mains-failure causing the stand-by generators to start up, the results might well have been catastrophic. For this reason the sheet piles were toed into firm material.

79. Continuous records of the reduced water level were kept as shown in Fig. 33. After first investigation it was thought that six wells would suffice; in boring these, the vagaries of the aquifer became revealed (Fig. 34); in all, eleven wells were sunk, nine being continuously pumped to yield 1500 g.p.m.

80. In reference to the Author's remarks in § 13 on supporting a cofferdam by berms or rakers, the cofferdam for the Houses of Parliament Boiler House in Black Rod's Garden in 1947–48 could be quoted. Where space allowed, this method was very economical of support timber or steel, and in excavation cost.

81. Although this cofferdam was within a stone's-throw of the River Thames, whose Mean Tide Level was about 0·0 Newlyn Datum, the groundwater level was -6 ft N.D. ± 12 in. The piles were 33 ft long, heads at -4 ft, 16 ft into clay, with toe level at 16ft below formation. Occasional pumping only was needed.

82. In 1959, about 200 yd from this site, the foundation for the tower block of the new Millbank development was taken down through peat and "bungum" into coarse gravel about 60 ft from the river foreshore. Tidal influence was felt up to $+2{\cdot}5$ ft; pile heads were set at $+4$ ft; the piles were driven an average of 6 ft into London Clay, but the cut-off here was not fully effective, either a slipped clutch or a pot-hole in the gravel made day-and-night pumping necessary, with a 3-in. pump. The supporting berm was in soft clay and peat, and began to dry out by exposure to the summer heat. Support of the piles depended entirely on this berm; to stop further drying out, it was parged over with mortar, and this quite effectively stopped any further desiccation.

Mr O. Kipps (Chief Civil Engineer, Holland & Hannen and Cubitts (Great Britain) Ltd) said that in § 5 the Author mentioned the development in the use of diaphragm walls constructed with the aid of bentonite in suspension, including their use on the excavations for the Hyde Park Corner under-pass, which was the first application of this technique in Britain. In addition to noise reduction and freedom from vibration which were important factors in congested urban areas, perhaps more interesting from the engineering point of view than these particular qualities were the advantages that this system of construction had to offer in overcoming difficult ground conditions: for example, the depth of penetration of driven steelsheet piling was normally limited to around 80 ft, but by the bentonite-suspension system the depth of penetration was theoretically unlimited. Certain mine shafts had been successfully sunk and lined down to more than 200 ft.

84. Another thing was that cobbles and boulders that would normally prevent the driving of steel piling or cause it to come out of interlock, as Mr Harding mentioned, were no deterrent to the diaphragm wall system, because the stones could be removed by the grabs or broken up by chisel. A positive penetration into rock could be achieved with the use of the chisel. On water-tightness, the use made of this type of diaphragm wall for permanent cut-offs should speak for itself.

85. A further point in favour was the inherent stiffness of a reinforced concrete wall. Normally it was 50 or 80 cm thick and this could simplify the problem of mechanical excavation methods beneath the struts. For structures like Fig. 2 the steel pile cofferdam could be replaced by a reinforced concrete in-situ diaphragm designed as part of the permanent structure with obvious possibilities of economy in construction time and cost. The slides showed the sort of work that could be done by this process. Figs 35 and 49 showed a cofferdam for the Quero power station on the Piave river in Italy which was similar to that in Fig. 22 of the paper. It involved a 50-cm. thick diaphragm wall penetrating 58 ft through sand and gravel down to schist bedrock into which the wall penetrated 5 ft. The groundwater level was at all times at or very near the surface of the excavation. The reason for adopting this construction was that this gravel

contained many large boulders which would have made it impossible to drive steel sheet piling.

86. Fig. 49 was the cofferdam during the excavation and it was very similar in style to that in Fig. 22.

87. Fig. 36 showed the cofferdam at Zevio power station, also in Italy, and rather like that in Fig. 9. It was 80 m by 120 m in overall size, penetrating to a depth of 20 metres, 15 of which were in sand and gravel, and the last 5 metres were in fine sand. The water level on this site was at or very near the ground surface all the time, and to relieve the pressure in the bottom relief wells were also used. Fig. 37 was another similar application, the intake for the Busche power station on the Piave river. In order to protect the foundations of the old bridge, at the same time as the excavation

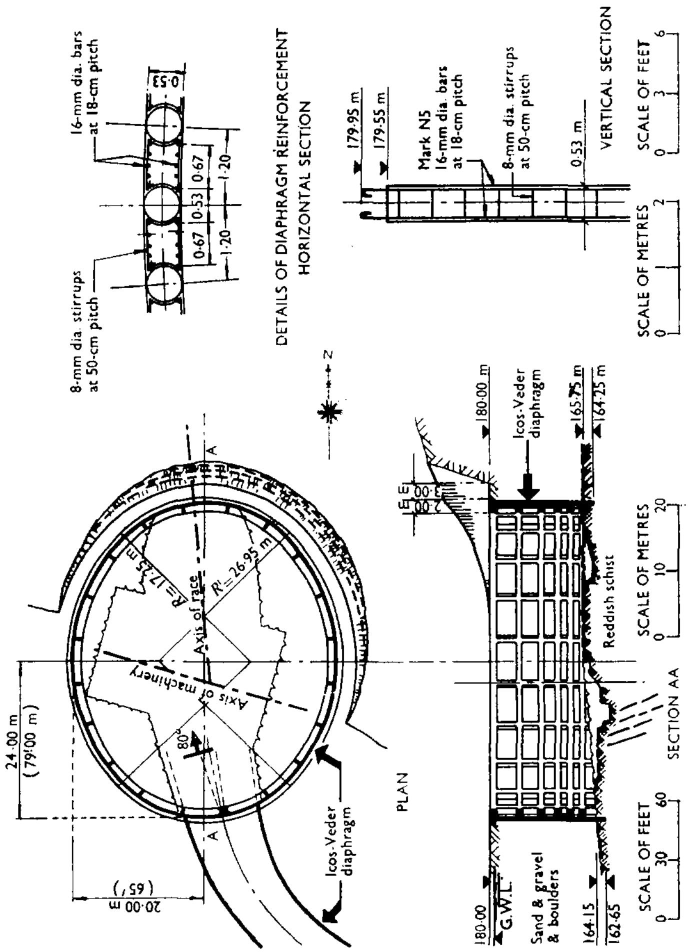


FIG. 35: COFFERDAM AT QUERO POWER STATION, PIAVE RIVER, ITALY

* Figs 49–54 are photographs and are placed between pp. 96–97.

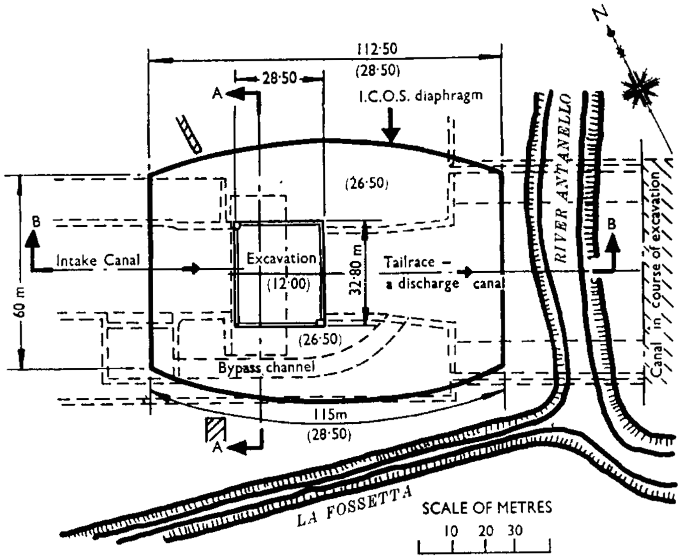


FIG. 36: COFFERDAM AT ZEVIO POWER STATION, ADIGE RIVER, ITALY

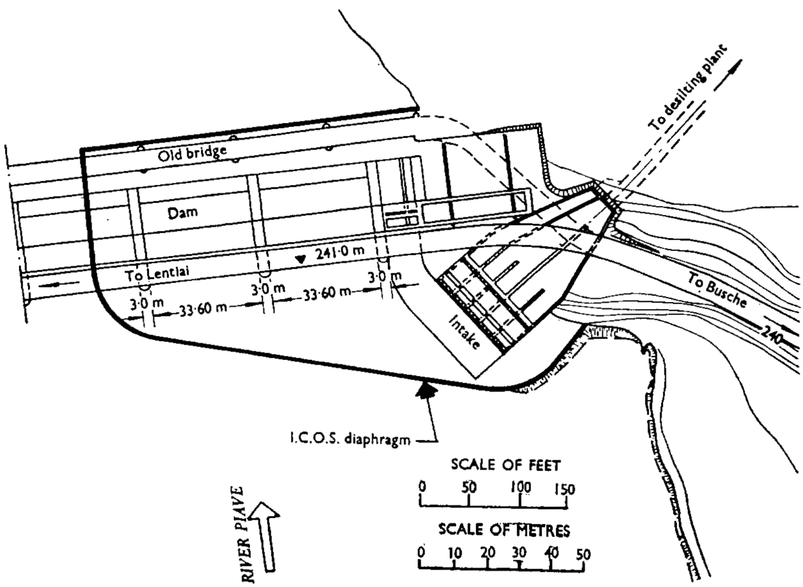


FIG. 37: COFFERDAM TO PROTECT INTAKE AND WEIR AT BUSCHE POWER STATION, PIAVE RIVER, ITALY

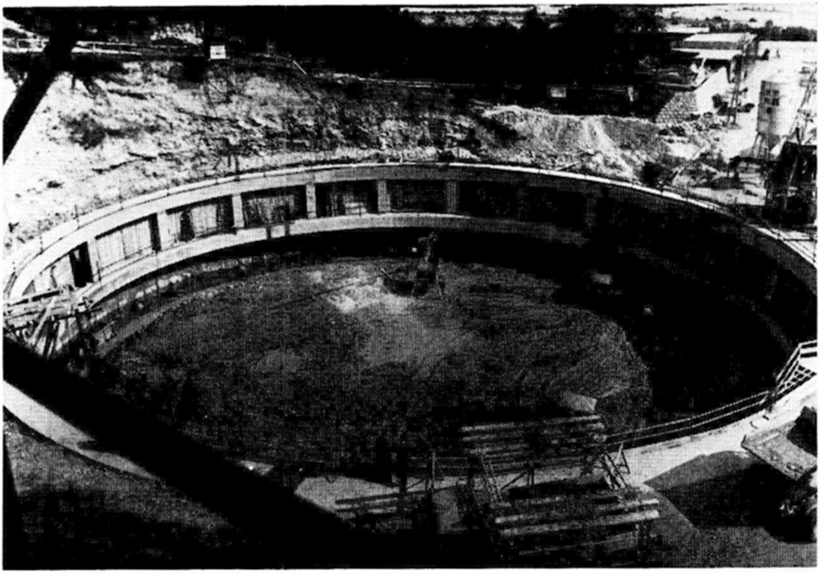
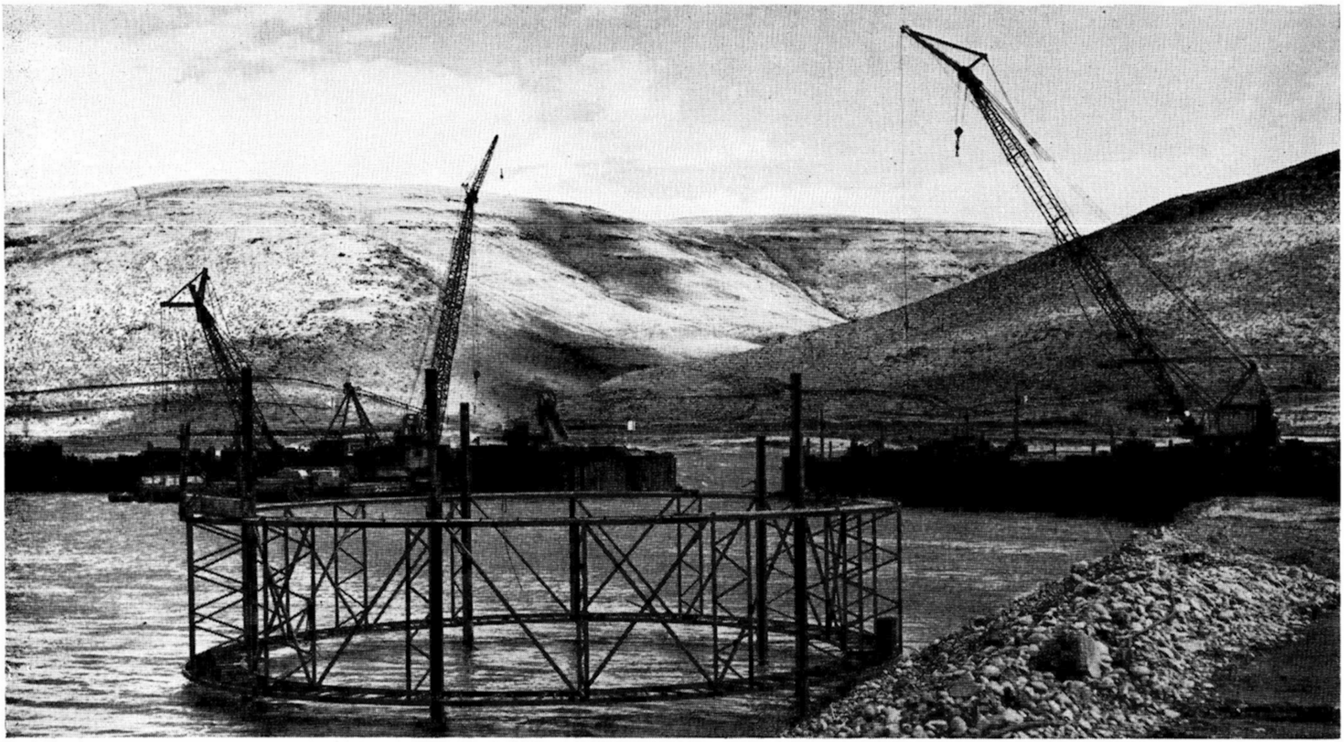


FIG. 49: COFFERDAM AT QUERO POWER STATION DURING EXCAVATION

FIG. 50: COFFERDAM PROTECTING FOUNDATION TO OLD BRIDGE AT BUSCHE POWER STATION INTAKE



(Courtesy "*The Em-Kayan*")

FIG. 51: JOHN DAY COFFERDAM

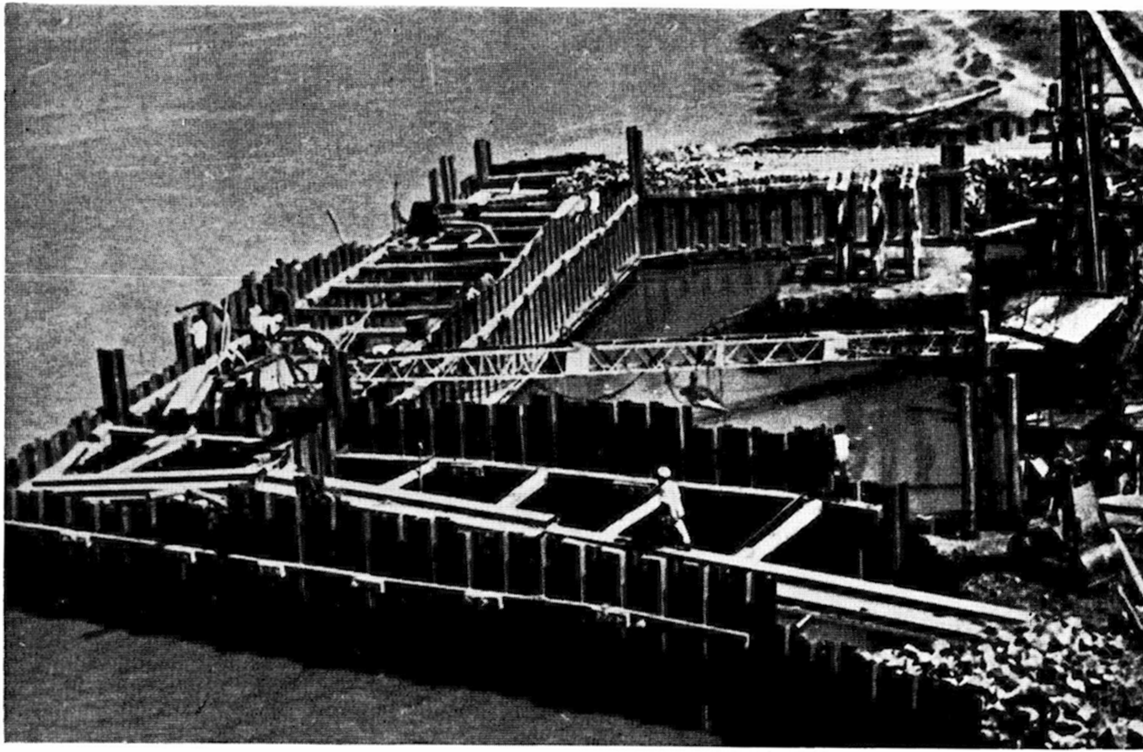
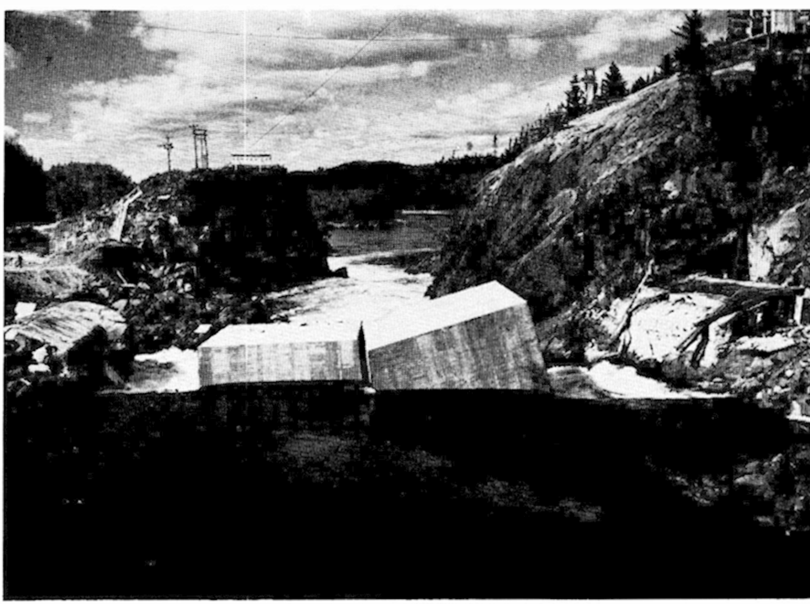


Fig. 52: Chola cofferdam, south India

(*The Shawinigan Engineering Co. Ltd, Montreal*)

FIG. 53: ST MAURICE RIVER, BEAUMONT DEVELOPMENT. UPSTREAM COFFERDAM, TIMBER CRIBS BEFORE LAUNCHING



(*The Shawinigan Engineering Co. Ltd, Montreal*)

FIG. 54: PERIBONKA RIVER, CHUTE À DIABLE DEVELOPMENT. TWO 2,600-TON MONOLITHS USED FOR RIVER CLOSURE

was taken out for the intake, the cofferdam was thrown right round a part of the existing structure as well as the site of the new intake; the reason for adopting this construction was the presence of many extremely large boulders deep down in the alluvial soils. With cofferdams of this type constructed in open water it was essential to throw an earth bund along the line of the cofferdam in the water before constructing the reinforced concrete wall. This type of construction had to start from an earth surface and could not be constructed from a free water surface.

88. Fig. 50 showed the cofferdam protecting the foundations of the old bridge.

89. It was a form of construction of which not very much had as yet been seen; all the examples shown were from practice in Italy; but it was one that should be used more in the right circumstances.

Mr D. A. Andrews (Civil Engineer, South Eastern Gas Board) said that Mr Packshaw's conclusions disturbed him a little because it seemed to him that however conservatively the engineer might approach the problem of design, his peace of mind would be in jeopardy if his cofferdams were of permanent construction. He wished to mention three gasholder tanks, two of which were constructed in the fashion of Fig. 13, with the principle in mind that it should be less costly to suitably seal the interlocks of steel sheet piling than to construct a prestressed or reinforced concrete wall.

91. Figs 38a and 38b were a 2·3-million cu. ft gasholder holding about 2·2 million gallons of water. It was constructed without difficulty and on first filling with water it lost about 38 000 gallons or 3½ in. per week. After the dumpling (the centre section) had taken up the water which had been drained from it during construction, the leakage improved to about 1 in. or 12 500 gallons a week. Every other pair of piles in the outer ring beam were driven lower to allow for easy fixing of the stirrups to the top ring beam and further investigation showed that the lifting holes in these pairs of piles were below the concrete ring beam and allowing water to escape. On sealing these holes the leakage was negligible.

92. Fig. 39 was for a slightly larger tank at Wandsworth, of 5 million cu. ft and holding 3·8 million gallons of water. Adequate site investigation could only be confirmed at the completion of any job and there was always the doubt if the job was a success that there had possibly been too much investigation. This holder occupied a square about 200 ft side, and five borings were drilled at approximately 100 ft centres and London clay was found at a constant level in all of them. Nevertheless a blow occurred at a position 30 ft from a boring where the top of the clay was found to be from 6 to 10 ft lower. On filling the tank for the first time about 25 000 gallons a day were lost. It had been their intention to keep the pits for the inlet and outlet pipes dry but it was decided at the last moment that they could be flooded. They had not, of course, made any provision for sealing the interlocks of the sheet piling in these pits so that a considerable quantity of this water was lost and on completion of the repair the losses were reduced to about 2000 gallons a day. At the same time the London County Council had constructed some sluices in the River Wandle which on being opened for the first time and causing the River Wandle to be tidal made the rate of loss from the holder tank go up to 5000 gallons a day, presumably by lowering the general water table. The position had now been improved and they were now only losing about 900 gallons a day.

93. Figs 40a and 40b were for a 2·6 million cu. ft gasholder holding just under 2 million gallons of water. Initially they had had difficulty in filling this tank because their supply was limited to about 20 000 gallons a day and they were losing about 50 000. They managed, however, to reduce the leaks sufficiently to keep the tank full for testing the holder. On completion of testing, the tank was emptied and it was found that the moat slab had parted from the inner ring of sheet piling. Repairs to this gap were in progress, and Mr Andrews could not say whether it was going to hold water on completion.

94. Generally as far as these structures exist as permanent works, he thought it a

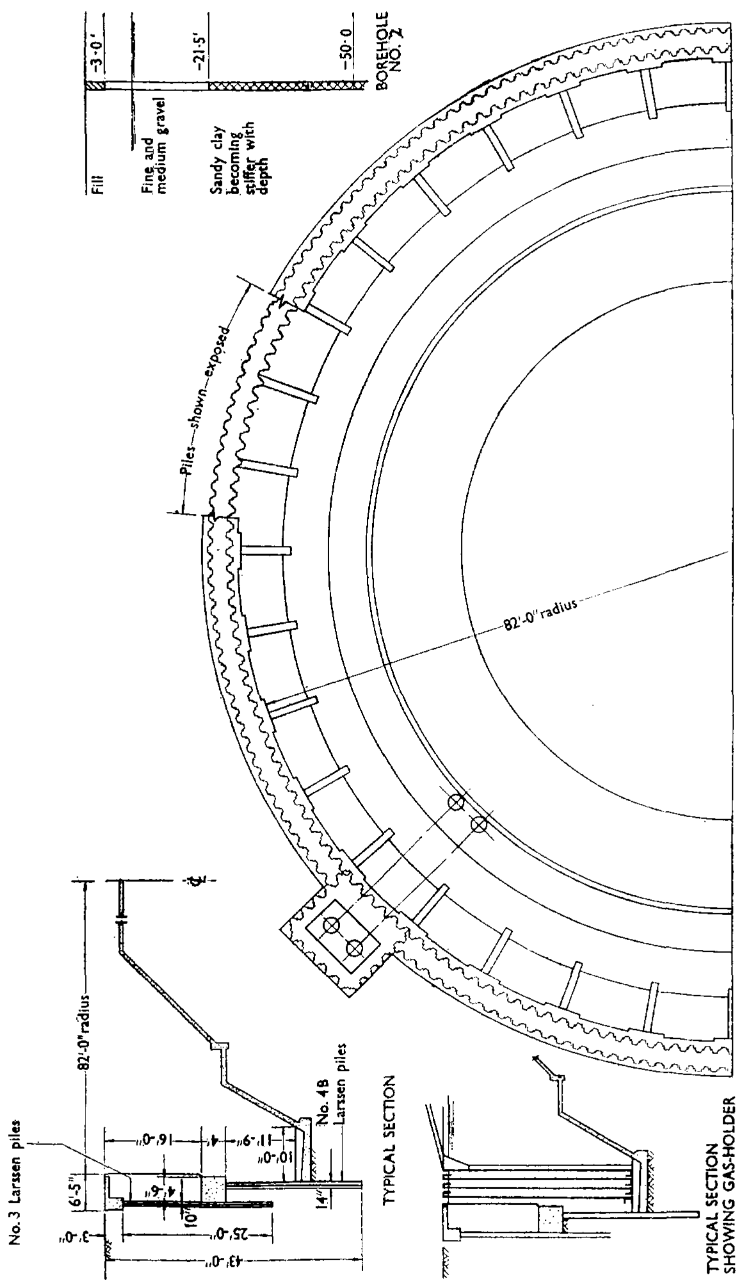
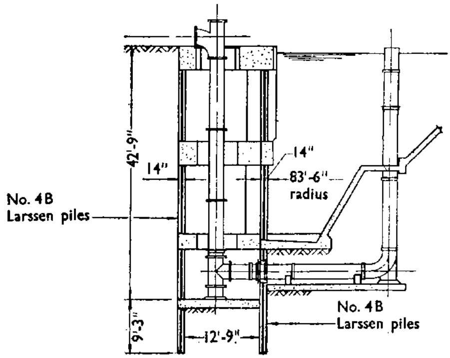


Fig. 38a: Sheet pile tank for 2·3 million cu. ft gas-holder, Kingston-on-Thames

fairly simple matter effectively to seal the interlocks of the sheet piling, but rather more attention should be paid at the junction of the piling with the concrete slabs and ring-beams.

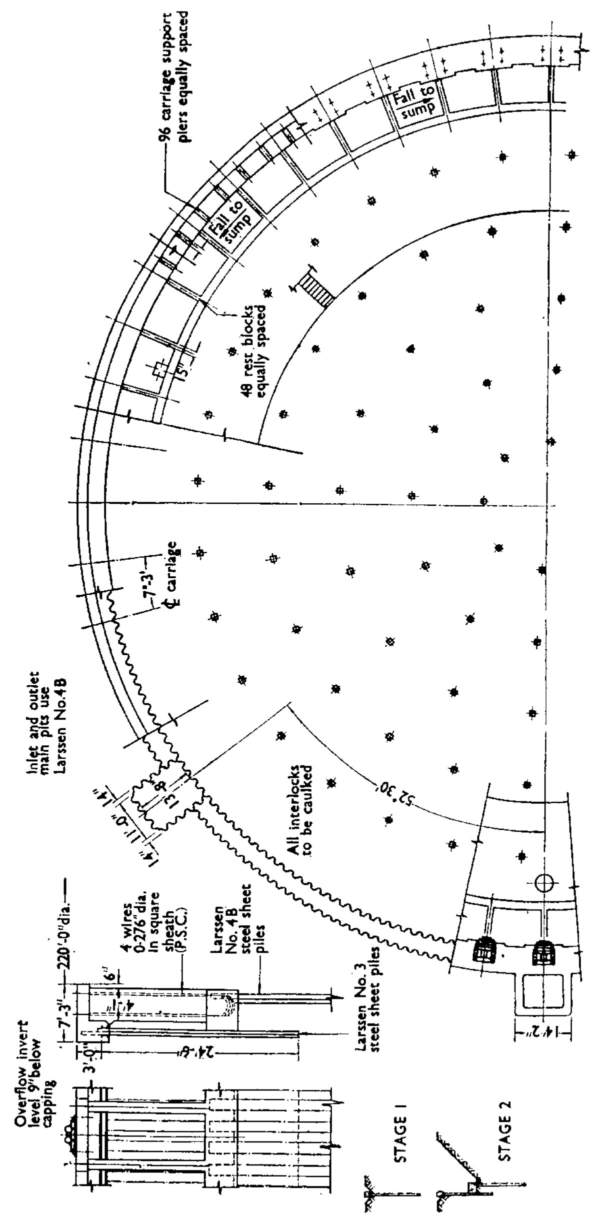


SECTION THROUGH PIPE PIT

FIG. 38B: ARRANGEMENT FOR INLET MAIN, KINGSTON GASHOLDER

Dr W. H. Ward (Head of Soil Mechanics Division; D.S.I.R. Building Research Station) said that it was discovered in the course of measurement by the Norwegians during the Oslo subway construction that one of the least suspected causes of very high loads on cofferdam struts was the action of frost penetrating through the walls during winter. This action was particularly important when the walls were of thin steel and in a cold climate. He had recently seen a lined cofferdam in Ottawa which consisted of two circular walls with a common chord of steel struts. This cofferdam was excavated to rock at 70 ft depth through a very soft sensitive clay and the diameter of the cells was about 60 ft. In the course of the last winter very high stresses had been measured in the upper 3 or 4 struts by the Canadian National Research Council and the struts commenced to buckle as a result of radial frost heaving around the wall of the cofferdam. The arrangements used to overcome it and also to provide a good deal of comfort for the workers was to hang a curtain around the whole of the inside of the cofferdam a few feet from the walls, and to provide a source of heat at the base so that hot air rose up between the cofferdam wall and the curtain. It had struck him as a very neat way of solving the problem. He also noted that this type of cofferdam provided a considerable amount of free working space compared with some of the cofferdams that Mr Collingridge had shown earlier.

96. He wished to comment on a remark Mr Packshaw had made concerning movements of the ground around cofferdams. There was always a measurable movement alongside any cofferdam even when the struts were preloaded during excavation. Lateral and vertical movements of the ground occurred as the excavation took place and while it was possible to annul the lateral movements by jacking considerable loads into the struts, it was never possible to avoid completely the vertical displacements. The latter movements were inevitable on account of the weight of ground removed in the course of excavation. The current work of constructing the road underpasses on either side of the escalator tunnel at Hyde Park Corner had already been mentioned by Mr Kipps. At this site B.R.S. had been able to measure quite accurately movements in the tunnel caused by the cofferdam construction on either side of it and at the present time everyone was having the greatest difficulty in keeping these movements within tolerable limits despite the fact that all the struts had been preloaded to some arbitrary values. It had been necessary to apply very considerable loads, corresponding to an


96 carriage support piers equally spaced
Fall to sump
Fall to sump
48 rest blocks equally spaced
7'-3'
℄ carriage
Inlet and outlet main pits use Larssen No.4B
All interlocks to be caulked
52°30'
220'-0" dia.
4 wires 0·276" dia. in square sheath (P.S.C.)
Larssen No.4B steel sheet piles
Larssen No. 3 steel sheet piles
Overflow invert level 9" below capping
24'-6"
3'-0"
14'2"
STAGE 1
STAGE 2

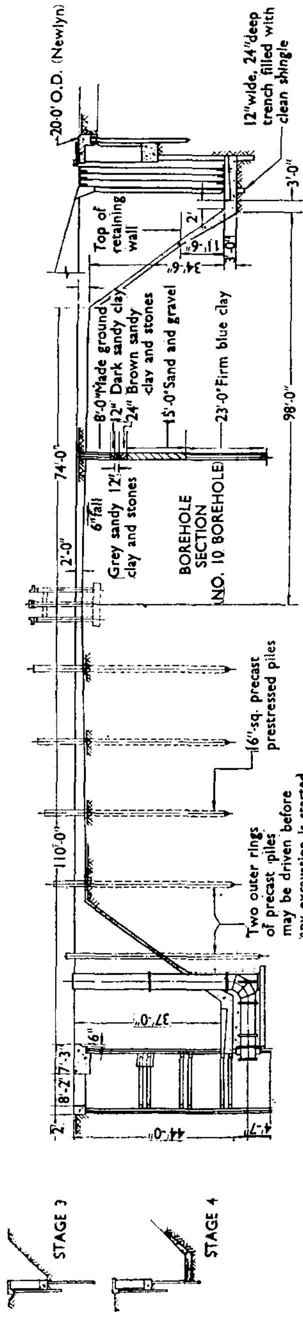


FIG. 39: SHEET PILE TANK FOR 5 MILLION CU. FT GAS-HOLDER, WANDSWORTH

(See contribution by D. A. Andrews for mention of Figs 39, 40a, and 40b)

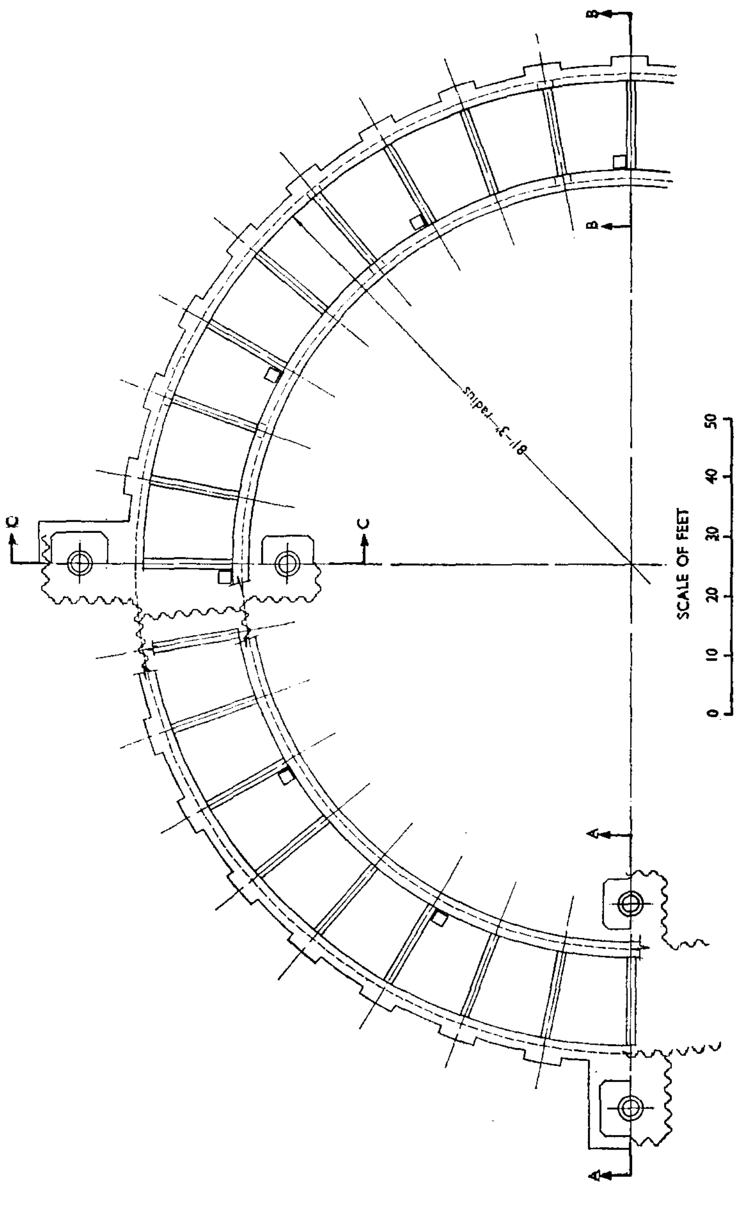


FIG. 40A: PART PLAN ANNULAR STEEL PILE TANK FOR 2·6 MILLION CU. FT GASHOLDER, GILLINGHAM

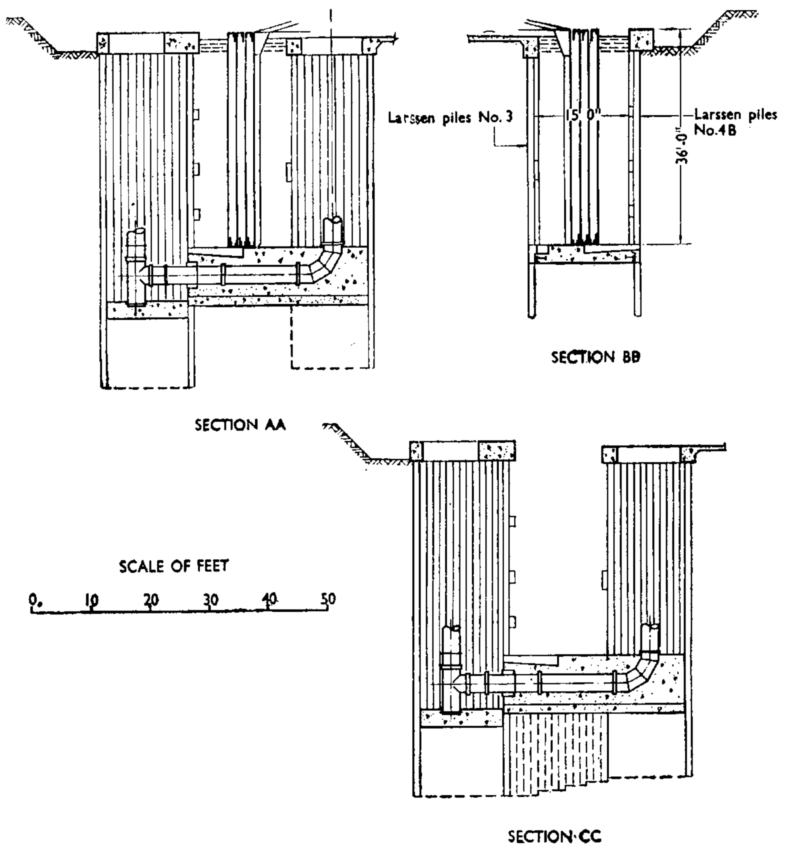


FIG. 40B: TYPICAL SECTIONS THROUGH ANNULAR TANK, GILLINGHAM

earth pressure coefficient of 2, to reverse the lateral movements which had occurred during excavation. It had become quite clear in the course of this work that the only satisfactory way of keeping a close control on the lateral movements of the ground was to observe them directly and to continue to increase the strut loads until the movements were sufficiently small.

Mr J. T. Edwards (Civil Engineer, Freeman, Fox and Partners, said that he wished to refer in particular to § 40 dealing with cofferdams in soft clay. Mr Packshaw had said that failures, particularly other people's, were always the most interesting! This was a failure or a partial failure of a cofferdam at a power station being built in Malaya. The site consisted of stratified soft, medium and firm white and brown clay, with bands of coarse sand and silt overlying deep granite. A number of bores were made on the site but none where the pumphouse was situated since its position was not then known. The samples taken in the clay gave shear strengths of 600 to 1500 lb./sq. ft and from those answers it was decided that whilst all the main foundations would have to be piled there would be no difficulty in constructing the cofferdam, which had to be slightly

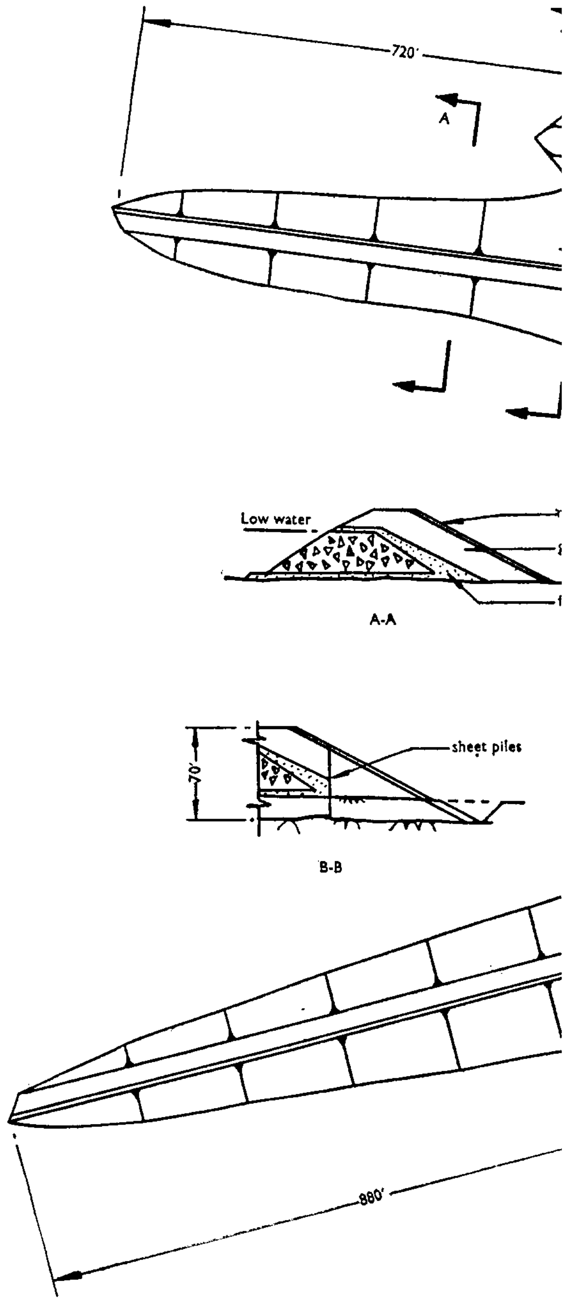
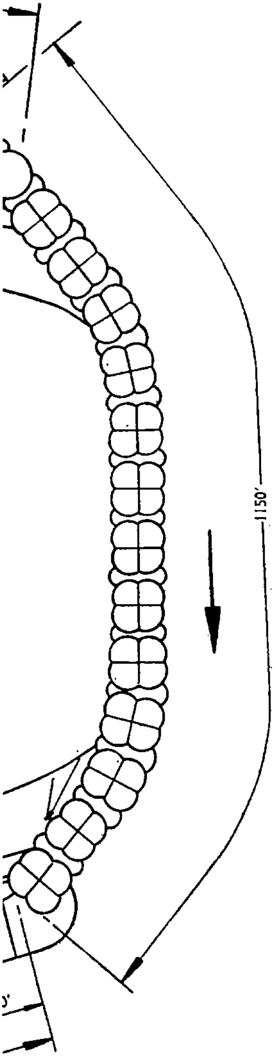


FIG. 45:



GRADINGS

Sieve (A.S.T.M.)	Per cent passing
Gravel	
No. 4	30% to 50%
No. 100	10% to 15%
Filter	
¾ in.	80% to 100%
⅜ in.	55% to 80%
No. 4	25% to 55%
No. 8	12% to 25%
No. 30	0 to 10%
No. 100	0 to 6%

Stabilized seepage
Normal 20,000 gal./min.
Flood 30,000 gal./min.
Pumping capacity 84,500 gal./min.

AY DAM

over 25 ft below ground level. The cofferdam was irregular in shape, 59 ft wide and approximately 100 ft long. The depth over most of the area and the area in which the failure occurred was 25 ft below ground level. The failure occurred along one of the shorter sides and was illustrated by three slides showing the sinking of the ground immediately outside the cofferdam, the movement of the piling away from the top waling and the heave in the bottom of the excavation.

98. It was considered that the failure was caused by soft clay in the foundation, similar to that described in § 40 of the Paper and in Reference 30. After the failure a number of bores were made. Those near the point of failure showed a remarkably uniform soft clay which, although it varied, had an average shear strength probably only of the order of 200 lb/sq. ft. Using Skempton's formula, the coefficient for a cofferdam of this length, width and depth was about 6 so that the critical depth was only about 12 ft. The irregularity of the strata accounted for the heave not occurring until a depth of 25 ft was reached. The clay which heaved into the excavation had been re-excavated and the area stabilized by driving a number of small timber piles, and the concrete had since been satisfactorily placed. The moral was that one could never have enough borings.

Mr T. R. M. Wakeling (Director, Foundation Engineering, Ltd), on behalf of himself and **Mr R. J. Hamilton** (Director, Richard Costain (Civil Engineering) Ltd), described a cofferdam which had been constructed in difficult ground conditions for the ash-disposal pit at West Thurrock Generating Station (C.E.G.B.). The consulting engineers were Sir Alexander Gibb and Partners.

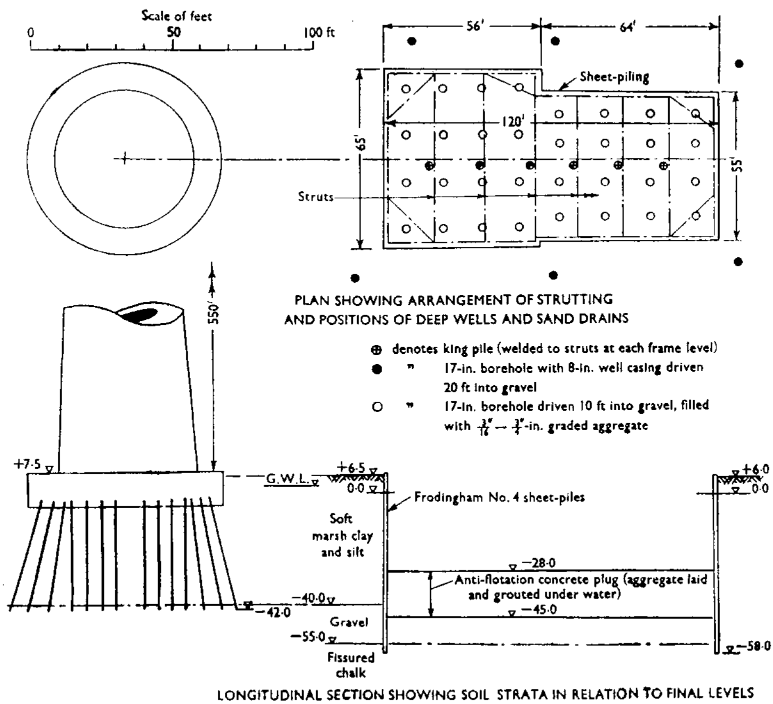


FIG. 41: EXCAVATION FOR ASH-DISPOSAL PIT

100. Fig. 41 showed the main dimensions of the ash pit and the soil conditions. The chimney on the left was 550 ft high in reinforced concrete and its construction had been well advanced before the ash-pit work had started.

101. It had been decided that the only practical solution was to construct inside a cofferdam, founding the ash-pit structure on the gravel about 50 ft below ground level.

102. The possibility of sinking a monolith had been ruled out because it would have needed complicated and massive internal bracing and because of the serious risk of undermining the chimney foundations if soil should blow in under the cutting edge.

103. The soft clay was impermeable and had a shear strength of about 400 lb/sq. in. The 15-ft bed of gravel contained water under a head of about 40 ft and rested on fissured chalk, which also contained water. It would thus be impossible to dewater the cofferdam completely, and it would also be necessary to guard against excessive loading on the piling from differential pressures during excavation.

104. Two facts had been clear from the outset: the work could not be done entirely in the dry; and the cofferdam would have to be strutted to as low a depth as possible.

105. The design had included three frames of strutting at levels $-2{\cdot}00$ ft, $-14{\cdot}50$ ft, and $-23{\cdot}50$ ft. Fig. 42 showed that there would then be 21·5 ft between the bottom frame and the bottom of the excavation.

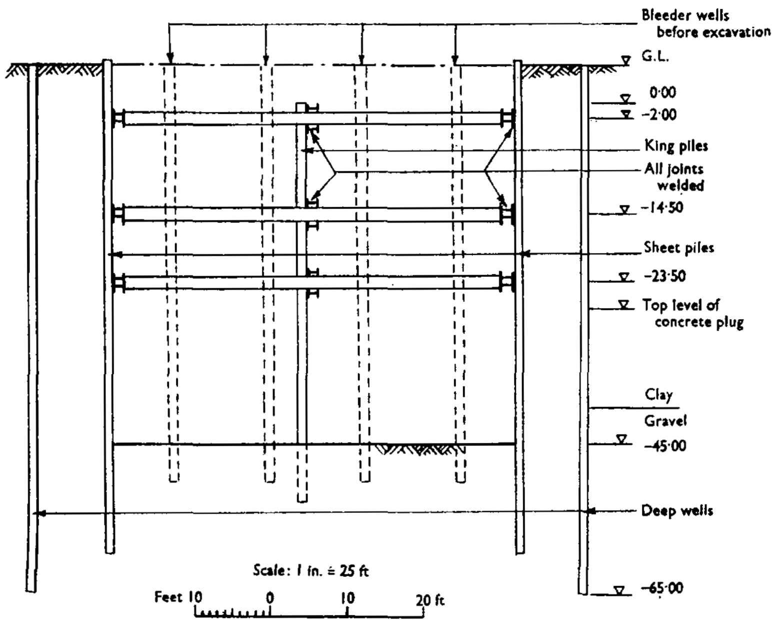


FIG. 42: TYPICAL CROSS-SECTION OF EXCAVATION

106. It had been planned to excavate to third-frame level in the dry, completing the excavation under water. There had remained the virtual certainty that the water pressure in the gravel would blow the last few feet of clay before the planned switch from dry to wet excavation could be undertaken. To guard against that, a series of bleeder wells had been sunk into the ballast throughout the ash-pit area on a grid of roughly 15-ft centres before the excavation had started. The bleeder wells consisted of 17-in.-dia. boreholes which had been backfilled with gravel as the drill casing was withdrawn.

107. After driving the sheet-piling into the chalk below the gravel and installing the bleeder wells, the first 12 ft of muck had been taken out and the first frame had been put in at level $-2{\cdot}00$ ft. The struts had been welded to the walings and the walings to the sheeting. The struts had been bracketed to a central row of Rendhex king piles.

108. Excavation had continued in the dry until the second frame could be installed at $-14{\cdot}50$ ft. Before the excavation reached the third-frame level, the bleeder wells, which had been discharging satisfactorily, had begun to show traces of sand and silt entering the pit. The cofferdam had been immediately flooded to equalize pressures.

109. In order to reduce the flow through the bleeder wells and the water pressure in the underlying ground, six drainage wells had been installed around the perimeter of the excavation, penetrating 20 ft below the clay. They were 8-in.-dia. perforated tubes surrounded by a 17-in.-dia. pack of filter gravel and fitted with submersible pumps. Before dewatering the dam, a trial run of the pumps had lowered the water table by 5 ft. After dewatering, the excavation had continued in the dry to third-frame level without further difficulty. The frame had been installed and the pit reflooded to level $-6{\cdot}00$ ft, the water being maintained at that level while excavating down to foundation level at $-45{\cdot}00$ ft.

110. The welding together of the struts, walings, and sheeting had been adopted to give monolithic rigidity to the cofferdam, as a safeguard against the very high clay and sand pressures at the lower levels. It had also avoided the need for diagonal bracing.

111. A 12-in. layer of coarse aggregate had been deposited over the entire base area to act as an absorbent layer, into which silt and fine sand could settle and form a seal against grout penetrating downwards. Three days had been allowed for settlement before the emplacement of the 17-ft layer of aggregate which was to form the anti-flotation plug. Grout-injection pipes had been positioned with their lower ends resting on the seal layer. The pipes had served at that stage to support vertical dowel bars for anchoring the final concrete flooring of the pit. The plug had been grouted under water by the Intrusion Prepakt method; after curing, the pit had been finally dewatered and the lower struts removed.

112. A 15-in. layer of blinding concrete and a 4-ft reinforced concrete topping slab, both anchored into the anti-flotation plug, had completed the bottom of the pit. The remainder of the construction had been in massive, but orthodox, reinforced concrete.

Mr I. B. Mackintosh (Senior Engineer, Taylor Woodrow Overseas Ltd) said that the Author had referred in § 15 to cofferdams built in a river usually on a rocky bed, and this was the problem which all dam builders faced. When one tackled large-scale work one needed to rely quite heavily on case studies. Unfortunately these had to come from overseas owing to the fact that there were no large-scale dam projects in Britain. The examples to which he would refer were from the Columbia river in the United States. The first two were not strictly cofferdam problems, although they involved closing off the river.

114. The Columbia river had flows of as much as 300 000 cusecs in the low flow period from October to February, and where the closure took several weeks a very large flow had to be designed for. This was so at McNary Dam (Fig. 43) where the river originally 2600 ft wide had been narrowed by successive cofferdams to a gap between two steel cells 240 ft wide. The water level had to be raised 17 ft in order to pass the river flow through the spillway blocks and that head drop caused a maximum water velocity of 30 ft/second. The method of closure was to dump 12-ton concrete tetrahedra by placing them into the flow from a cableway. This work was done nearly 10 years ago since when other shapes have been developed which were much superior for this purpose and so tetrahedra would not be used again.

115. The next example was The Dalles Dam (Fig. 44) where a dumped rockfill was used for closing the 500 ft wide channel and this material was incorporated in a permanent rockfill embankment 285 ft high. At the deepest point the river bed was 180 ft below low water level and as a preliminary operation in an earlier season this depth

was reduced to 55 ft by dumping rock from barges. The first stage of the final closure was done by end-dumping rockfill on a frontal width of 240 ft, working out from the right bank until the river had been closed to a channel width of only 25 ft. The water velocity was then 12 ft/second. At first rock lumps averaging 500 lb in weight were used but it was later found that this could be reduced to 200 lb. The rockfill was followed by end-dumping smaller rock and filter, and then a sand blanket, part of which was lowered in place from barges. The final closure operation was carried out in a period of 1½ hours by concentrating all available equipment on end-dumping on a 60 ft frontal width. The reason for the short time was to have the river shut off before the flow in the river was able to build up the head.

116. John Day Dam was the latest project on the Lower Columbia river and it would have an ultimate capacity of 2 700 000 kW making it the largest power station in the United States. The first stage cofferdam (Fig. 45) enclosed the spillway blocks. Most of the cofferdams on other projects on the Columbia River had been cellular

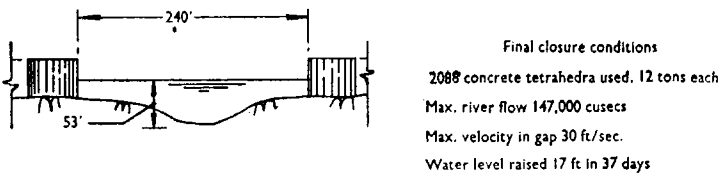


FIG. 43: MCNARY DAM

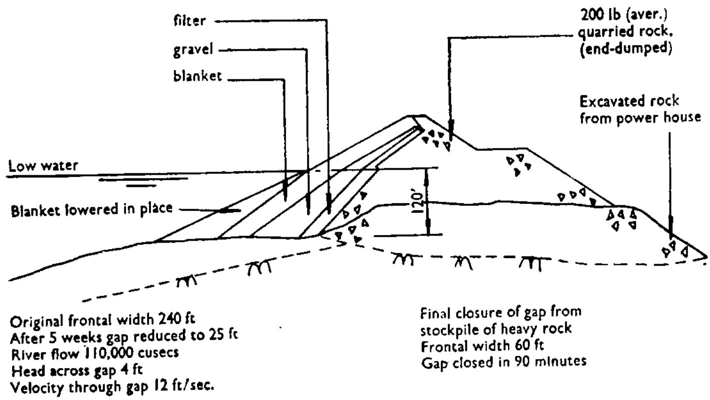


FIG. 44: THE DALLES DAM

cofferdams throughout their length. At John Day for reasons of economy the upstream and downstream wings of the cofferdam were rockfill dykes, and steel cells were only used in the part parallel to the river where erosion would be a maximum. After dredging 20 ft of river bed gravel to expose bedrock the clover leaf steel cells were constructed by driving the piles a foot or two into the surface of the rock working from floating craft (Fig. 51). Divers then went down into the cells to inspect the bottom and after loose material had been removed with an air lift the divers placed bags of sand and cement along the inside periphery to prevent loss of fill material and to reduce leakage under the piling. The bags were placed in 3 rows, 6 bags high against the piling and stepping down to 4 bags high and 2 bags high on the inside rows. Following this operation the cells were filled with free-draining gravel. The maximum height of the cells was 80 ft and the weight of piling used was 6000 tons. The cross section of the rockfill dyke and the specifications for the gradings of the various zones was shown (Fig. 45). A pumping capacity of 84 500 U.S. g.p.m. was required for initial dewatering but when the cofferdam had become stabilized by silting up of some of the finer paths, the steady seepage was about one quarter of this at low water. This cofferdam was built by a separate preliminary contract worth $1,600,000 and the contract period was 12 months.

117. Firms tendering for large overseas dams had to use these case studies in preparing their tenders. A very interesting cofferdam was required for the Volta Dam where a rockfill cofferdam similar to the one at the Dalles had to be constructed in a 200-ft depth of water. As an unsuccessful tenderer for that work he wished the Italian contractor the best of luck for the successful execution of this enormous cofferdam.

Mr L. G. Ellis (Resident Engineer, Medway Bridge; Freeman, Fox and Partners) described a design done twelve years ago for temporary work connected with an Indian power station. This temporary work had been constructed to enable a cooling-water pumphouse to be built in a dry unrestricted area enclosed by a double-walled cofferdam.

119. At this time the codes of practice for foundations and earth-retaining structures had not yet been published and among his researches none was more helpful than the Author's contribution to the discussion on the paper under reference 31. There was also available the interesting example of the double-walled cofferdam at Carrington (Fig. 26) which he had visited.

120. He was surprised at the narrow width of the Carrington cofferdam until he heard of the considerable penetration of the sheet piling. He wished to ask the Author whether a wider cofferdam would not have resulted in a more economical length of piling, and whether the chosen design was intended to act partially as a twin cantilever sheet-pile retaining wall.

121. Since the publication of the Author's Paper he had studied the references to double-walled cofferdams in the bibliography and it usually appeared that a considerable penetration of the walls had been commanded even when the sheet piling had to be driven 4 to 5 ft into rock.

122. The Code of Practice for Foundations stated that double-walled cofferdams were not suitable where the piling could not be driven into the soil, for instance on hard rock, unless some means could be adopted for securing the bottom of the sheeting against the outward pressure of the filling between the two lines of piling.

123. Having regard to the expense of driving sheet piling any distance into rock, the waste in damaged piling after extraction, and the requirements of the Code of Practice, he wished to describe briefly the design and construction of a double walled cofferdam founded on rock without any appreciable penetration of the sheet piling into it.

124. The overall design was part empirical but the cofferdam was considered for sliding and overturning as a gravity structure, as an inner and outer wall acting as a retaining wall and anchor wall respectively, and the section parallel to the river was also considered as a thick rectangular slab, lower end fixed, vertical edges freely supported, upper edge free. In addition shear on a central vertical plane was considered for the most heavily loaded section.

125. Fig. 46 illustrated two of these design conditions; shear on a neutral plane, considering a linear stress distribution on the base in opposition to a frictional force arising from the constraint offered by the contained materials; and an evaluation of tie rod loads based on the active pressure diagrams for both walls. The 2 ft 6 in.-thick concrete slab was cast in sections in trench after dewatering but before general excavation commenced.

126. The river was tidal, subjected to flooding during the monsoon, and it was always

envisaged that at some time the whole of the temporary works might be completely submerged. For this reason the cofferdam was also considered for sliding and overturning as a gravity structure acting in the opposite direction, assuming water locked up in the area enclosed by the cofferdam at the level of the top of the sheet piling, and low water level outside the cofferdam. The sheet piling, walings, and tie rods were also designed for their maximum loading which resulted when the cofferdam contained saturated filling after the area enclosed by it had been pumped dry.

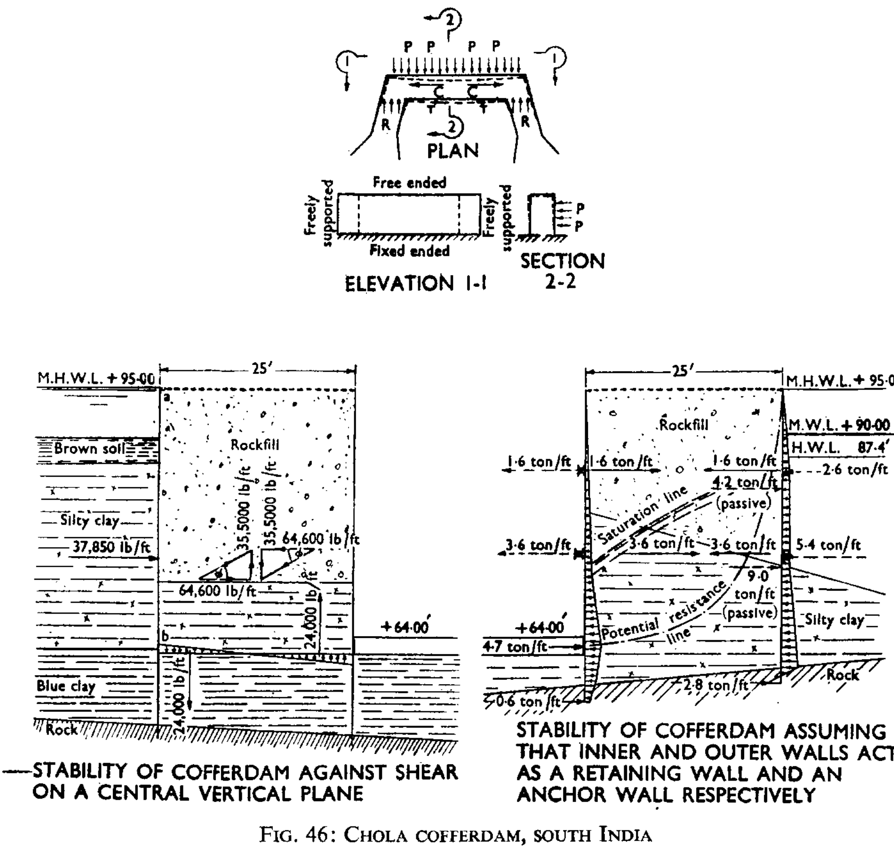


FIG. 46: CHOLA COFFERDAM, SOUTH INDIA

127. Fig. 52 showed the cofferdam under construction and illustrated the continuous steel walling, the temporary timber strutting, the steel tie rods, filling being imported, and the steel sheet pile bulkheads at both ends of the river section. Sluice valves were provided to equalize water levels inside and outside the dam prior to its complete submerging at maximum high water level. Normal high water level was approximately 7 ft 6 in. below the top of the sheet piling but the cofferdam was nevertheless designed for overtopping.

128. This proved a wise precaution, because the day arrived when the workmen left without making provision for operating the sluice valves. A monsoon caused the

tide in the river to rise to the top of the sheet piling and the cofferdam was duly overtopped. However he was pleased to report that neither the design nor the construction of the cofferdam had been discredited by this test load.

129. In the bibliography and in the brief description in the Code of Practice he had noticed many references to weep holes. Every permanent retaining wall which he had been associated with during the last ten years, often far away from water, had been provided not only with weep holes, but with an interconnecting system of drainage in the filling behind it. He considered this equally important in a double-walled cofferdam.

Mr W. J. H. Rennie (Director, Charles Brand & Son Ltd) observed that Mr Packshaw had limited his studies to 1940 onwards and in doing so had made a number of important omissions.

131. No reference was made to the cofferdams and Sudds associated with the construction of the many dams and barrages on the River Nile, where he had spent a number of years, and which he felt merited some mention if only for their magnitude and complexity.

132. Two types of cofferdam, not mentioned in the Paper, were used on the construction of a dam on the River Perak in Malaya[36].

133. The flood conditions of the River Perak were such that although comparable in magnitude to the Nile at Aswan or the Zambezi at Kariba, they did not have the seasonal cycles of these rivers, and flooding in Malaya could occur at any time of the year. The cofferdams, therefore, had to be designed and constructed to be overtopped because it was considered to be neither practical nor economical to make provision for excluding the maximum floods.

134. Mr Rennie showed several slides of the dam at various stages in the construction to illustrate the A-frame type constructed on the irregular rock surfaces of the river-bed to form the river arms of three of the main cofferdams. The final closing of the river was made with the crib and stop-log type and here the methods and sequence of operations of the construction of this cofferdam were described and illustrated.

Dr A. W. Bishop (Reader in Soil Mechanics, Imperial College of Science and Technology, London) said that in his conclusions the Author had stated that in the present state of knowledge it was necessary for cofferdams to be conservative. However, even to be sure of being conservative one had to have some quantitative basis for design.

136. The Author had suggested in § 68 that if failure did occur it was almost invariably due to base failure in soil. Yet in his treatment of double-wall cofferdams in § 52 the permissible width/height ratios (not less than 0·7 to 0·8) were apparently largely controlled by the shear strength of the filling, though no quantitative relationship was suggested.

137. In 1952 Professor Skempton and Dr Bishop were asked to advise on the design of a large double-wall cofferdam for the entrance to the Gallions Lock, which was then being rebuilt[37]. A search of the literature revealed no rational method of design which took both the strength of the fill and the strength of the subsoil into account. A method of analysis was therefore devised (which could probably be improved on after 10 years) and the results given by it appeared to suggest that Mr Packshaw's proposed ratios of 0·7 to 0·8 might under some circumstances not be conservative at all.

138. The cross-section of the cofferdam was illustrated in Fig. 47. The analysis was based on the following soil properties:—

Foundations: soft chalk:

Undrained shear strength	
(*a*) upper 3 ft	10 lb/sq. in.
(*b*) average of upper 10 ft	18 lb/sq. in.
Density (submerged)	60 lb/cu. ft

Fill: hardcore or gravel:

Angle of friction ϕ'	35°
Angle of wall friction on steel piling δ	18°
Density (above water level)	115 lb/cu. ft
Density (submerged)	65 lb/cu. ft

139. The thin layer of natural gravel overlying the chalk was assumed to have the same properties as the fill. The value of the angle of wall friction was conservative and might in practice be as high as 26°.

140. On the basis of the values tabulated, the factor of safety against sliding in the upper layer of the chalk lay between 1·4 and 1·6 depending on the strength assumed for the old concrete sill. The sheet piling, being relatively weak in bending, made only a small contribution to the factor of safety.

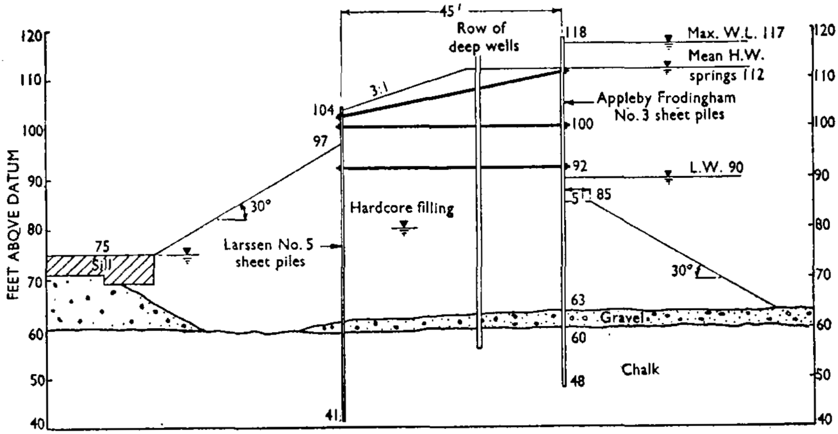


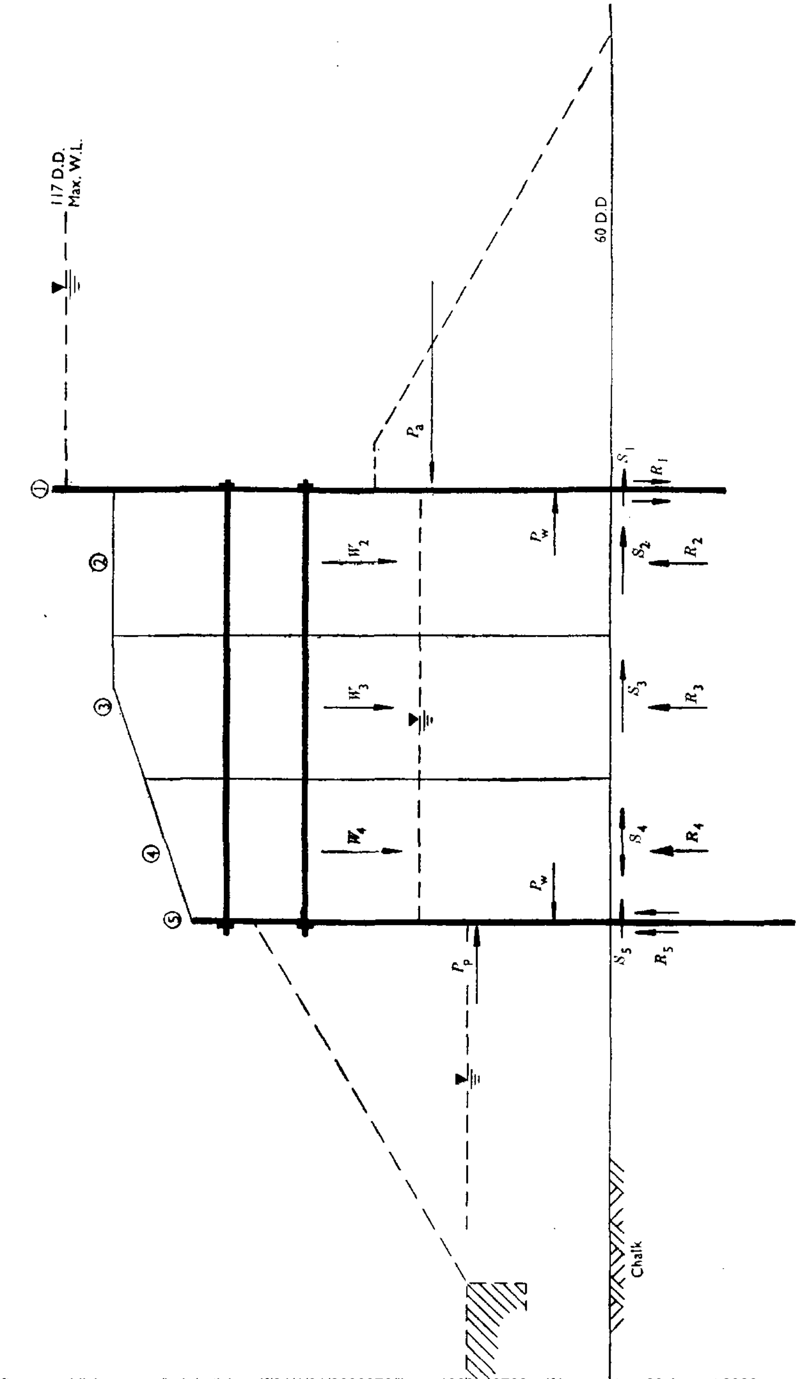
FIG. 47: DOUBLE-WALL COFFERDAM. GALLIONS LOCK OUTER ENTRANCE

141. Shear failure of the fill within the cofferdam would lead to the cofferdam collapsing backwards at high tide like a row of unsupported books subjected to a lateral thrust. For the purpose of analysing this type of failure the cofferdam was divided into five elements separated by vertical boundaries on which the magnitude of the shear forces could be determined (Fig. 48)—the outer piling, three slices of the hardcore filling and the inner piling.

142. The total thrust P_a on the outer piling was that due to water pressure and the active pressure of the riverside berm. The resistance supplied by the passive pressure of the inner berm P'_p and by the water standing behind the cofferdam was denoted by P_p and the probable levels of the resultant forces were determined.

143. The problem was then to find what factor of safety had to be applied to tan ϕ' and tan δ, and to the passive pressure P'_p, to satisfy all the conditions of equilibrium of the five adjacent elements of the cofferdam. The solution was obtained by successive approximations, the force polygon corresponding to one set of assumptions about the distribution of base shear being illustrated in Fig. 48. In this particular case the contribution to base shear of the front row of sheet piling was taken to be zero. The resulting factor of safety was 1·4.

144. With the assumption that δ might be as high as 26°, and with two alternative distributions of base shear, values of the factor of safety of 1·5 were obtained. These



FIGS 48A, B, & C: STABILITY ANALYSIS OF GALLIONS LOCK DOUBLE-WALL COFFERDAM values could hardly be called over-conservative and in fact the top of the riverside sheet piling was deflected about 14 inches after it had been subjected to the full water load. Yet the width to height ratio as defined in § 52 was rather more than 1·0 and supporting berms were used.

145. It would be noted that the tie bar pull did not occur as a term in the analysis. The action of the water pressure on the outer sheet piling would be to reduce this term to a small value, and it was on the safe side to omit it in examining the factor of safety

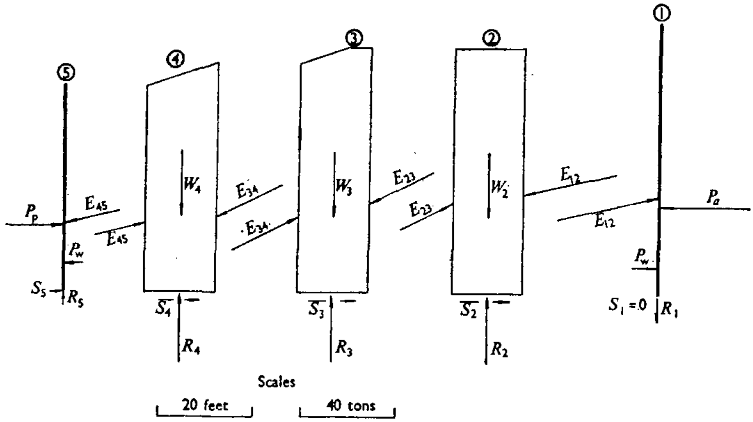


FIG. 48B

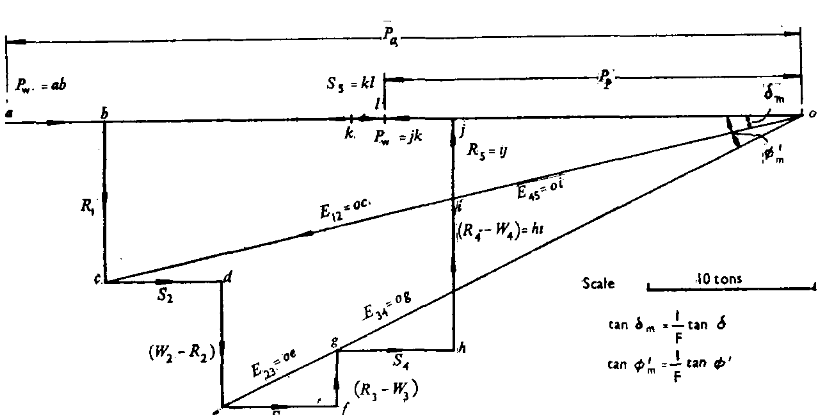


FIG. 48C

against shear between the vertical elements. Only if shear strains corresponding to large angular distortions were permitted would its value be expected to increase due to the dilatancy of the hardcore. It was at low water level in the river that the maximum tie bar pull occurred and it was most important to hold the water level down inside the cofferdam to keep this pull, and the bending moment in the outer piling, within reasonable limits.

146. An interesting example of the collapse of a cofferdam as a result of a structural failure had been described by Professor Tschebotarioff[38] at the Conference on Earth Pressure Problems at Brussels in 1958 and should no doubt find a place in the Author's valuable collection of case histories.

The following contributions were received in writing:

Mr George Hedley (Civil Engineer, Borough Engineer's Department, Newport, Mon., previously at Weston-super-Mare) wrote that he wished to prevent a misunderstanding concerning Fig. 2 and the alternative design Fig. 13.

148. To avoid difficulties arising through the use of negative levels he had fixed an arbitrary datum at 100 ft below Ordnance Datum. The levels shown by Mr Packshaw referred to this arbitrary datum.

Mr G. M. Cornfield (The British Steel Piling Company Limited) wrote that he would like to attempt to summarize the position regarding existing knowledge of the design of steel-sheet pile land cofferdams of the single-skin braced type. Generally it could be stated that the design problem was complicated because the usual Coulomb or Rankine earth-pressure theories were not always strictly applicable owing to the deformation conditions, and also because of the transference of soil load from one strut level to another via the sheet piling due to the deflexion of the piling which had occurred before placing each bracing frame in position.

150. He would consider four different basic soil conditions which might exist over the depth of the cofferdam, thus: (1) Dry cohesionless soil; (2) Cohesionless soil with a high groundwater level; (3) Soft saturated cohesive soil; (4) Stiff saturated cohesive soil.

Dry cohesionless soil

151. For this soil condition a number of cases[26, 39, 40] were recorded where field and model measurements had been made, and simple design procedures were given in Refs 19 and 37 for estimation of strut loads, making use of an empirical trapezoidal soil pressure distribution. However, this condition was very infrequent in practice.

Cohesionless soil with a high groundwater level

152. After an extensive search through the literature he had found not even a single reference where field measurements had been recorded for this condition. There was no theory which was really applicable, and there was doubt regarding the applicability of the empirical methods in Refs 19, 39, which apparently referred to dry cohesionless soils only. Though the pressure of the water contained in the soil would have a triangular distribution increasing with depth, he believed it was possible that the loading on the struts due to the water pressure would not correspond to such a distribution, and there might instead be a trapezoidal distribution of strut loads due to shear transference of load through the piling, as mentioned earlier. (In Reference 22, though the soil was a soft clay, it was shown how an approximately triangular soil pressure distribution could result in a final trapezoidal distribution of strut load.) Otherwise one was left to make the crude assumption of triangular distribution of both soil and water pressures for estimation of strut loads.

153. This unsatisfactory position was aggravated by further factors if the sheet piling did not penetrate to an impermeable layer below the excavation level. If pumping was carried out from sumps, the seepage of water would have the effect of altering the net static water pressure distribution, particularly below excavation level. However, the seepage forces also resulted in a small increase in the unit submerged soil weight on the "active side" of the piling and a very large reduction in this weight on the "passive side", depending on the extent of the penetration of the piling below excavation level[41, 42]. The resultant effect was an increase in the bending moment on the piling below the lowest bracing frame and a significant increase of the loads on this frame due to the reduction

in passive pressure, as compared with calculation based on no seepage. If pumping was carried out by borehole pumps located inside the cofferdam with the pumps at a level near to the toes of the piles, it would be possible to eliminate most of the effects of seepage, and in addition to avoid the possibility of piping.

Soft saturated cohesive soil

154. There were a number of references available (22, 23, 24, 26, 39, 43–46) giving details of field measurements which had been carried out for this condition, though in nearly all these cases loads on walings or struts had been recorded rather than pressures on the sheet piling. A point to be considered in the interpretation of the results was the very high scatter of the readings. (Also in some of the recorded cases[44–46] the results were complicated by possible soil freezing effects due to low temperatures during construction, and also by the prestressing of struts.) It had been suggested[22] that the pressure on the piling was in fact given by normal earth pressure theory for this soil condition, but that shear transference caused by deflexion of the piling resulted in a trapezoidal type of distribution of the waling loads of the various frames.

155. Based on such field measurements, two empirical design procedures had been proposed for estimating the waling loads; first, that usually referred to as Peck's method[39, 43], and secondly that given in the Code[19]—Earth Retaining Structures, p. 29. In both of these, classical earth pressure theory was used as a basis for obtaining an empirical trapezoidal diagram of pressure intended for the calculation of the waling loads.

156. Application of this Code method to a number of the cofferdams where measurements have been recorded (for example Refs 22, 23, 24) indicated that it gave results which were on the safe side and amply covered the maxima of the widely varying loads which were measured. The procedure given in the Code was intended to be applied for the final condition of the cofferdam, i.e. when excavation had reached the lowest level, and therefore some care must be taken to ensure that an unsafe condition could not occur during excavation and placing of the frames. One point on which the Code did not give any guidance in application of this method was whether account should be taken of any net active pressure below excavation level.

Stiff saturated cohesive soil

157. This last condition was difficult for two reasons. First, even assuming normal earth pressure theory was applicable, theory indicated zero pressure on the piling—unless the doubtful assumption were made that the water contained within the body of the clay could act separately and exert pressure on the piling. Second, he could find only one recorded case[47] where site measurements of strut loads were made, and even this was on a timber-runner-sheeted trench, and there was a very large scatter of the results. Field measurements had been recorded elsewhere[13, 48, 49] but in the last two the soil was a non-saturated clay, and in the first the bracing was jacked until an "at-rest" pressure condition was obtained.

158. Reference to the Code[19] Earth Retaining Structures indicated three alternative possibilities. Firstly, that the soil pressure should be taken arbitrarily as that due to a fluid having a density of 30 lb/cu. ft, and that this was used as a basis for obtaining the trapezoidal diagram in accordance with the procedure stated in the Code; secondly, that tension cracks could occur between the sheet piling and the soil, and that these would be filled with water; thirdly, softening of the clay could occur with time and a lower softened shear strength should be used. He believed that the second and third possibilities would not apply to normal temporary braced cofferdams, and that the first method could reasonably be applied until further field evidence was available.

159. It would be seen that there was a particular need for further field experiments to be carried out on braced sheet pile cofferdams, especially for the soil conditions (2) and (4) of § 150, where available theory and knowledge were very limited. In spite of this, cofferdams must still be designed and constructed, but the factor of safety might be

unnecessarily high in some cases and too low in others. Careful judgment and experience were necessary if these extremes were to be avoided.

Mr John R. Russ (Field Engineer, The Shawinigan Engineering Company Limited, Montreal) wrote that it was very difficult for an outsider to judge the merits of a cofferdam without the following information:—

(*a*) The precise nature of the foundation.
(*b*) The general design of the cofferdam.
(*c*) The resulting leakage and any other pertinent performance characteristics.

161. In a general paper such as Mr Packshaw had presented, it was obviously impossible to give detailed case histories.

162. However, it would have been of interest if more details of the foundation material had been given on the many drawings printed. For example, in spite of reading Mr Packshaw's Paper, the original Paper (No. 6479) on Kariba and also the recent discussion on this Paper, the writer was still unable to form an idea as to the exact nature of the alluvial deposit which formed the foundation of this cofferdam.

163. Admittedly, many cofferdams were built without previous investigation of the alluvial deposit. However, where high heads were involved, a knowledge of the grading and porosity of the foundation material became essential.

164. When building very large cofferdams, such as were common in hydro-electric work, dewatering and maintenance pumping costs could form a high percentage of the total cost. Ackerman and Locher in "Construction Planning and Plant" quoted the cost of power alone for pumping at Bonneville Dam as $50,000. On a comparable development in the writer's experience the cost of dewatering and maintaining the Stage 1 cofferdam amounted to over $300,000 for a six-month period. Thus the success of a cofferdam depended not only on its stability but on its watertightness also.

165. Mr Packshaw stated that when dewatering with a constant outer water level "ample pumping capacity must be provided". Was he able to suggest any working rules for estimating the capacity required? For a contractor this might prove an important problem. If he installed too many pumps he wasted money; if he installed too few he might waste several weeks before he gained control of the situation.

166. Crib-fill cofferdams were very popular in the Province of Quebec, where relatively cheap timber and rocky river beds made their choice a logical one. The local method of constructing these dams might be of interest since it was a very quick one and in cofferdam construction speed was usually of prime importance. The cribs were built upside down with the tops tailored to the contour of the river bed. When it was large enough for launching, the crib was tipped head over heels into the river. (Very large units could be dealt with in this way as shown in Fig. 53. The pocket sizes in the picture were 8 ft × 8 ft and the timbers were 10 in. × 10 in., making the total length of the cribs 56 ft and 64 ft. The maximum depth of water here was 40 feet and the flow was about 20 000 c.f.s.). Two or more pockets were blanked off near the bottom with heavy timbers, usually 8 in. × 8 in. or 6 in. × 6 in. and these were then loaded with rock to sink the crib. If the water was deep the crib was extended as it was sunk. If the water was fast flowing, the cribs were handled by cables from upstream.

167. The rate of construction was high. In normal conditions, a gang, which was composed of about 15 men, would build 100 cu. yds in a 10-hour shift. In a fast cross-current (say 7–12 m.p.h.) or in very deep water (over 35 ft) this rate would drop to about 75 cu. yds per 10-hour shift. These rates included the loading of the cribs but did not include sheeting or sheet piling as the case might be, nor toe-fill. Once above water level the working rate jumped to around 400 cu. yd per shift per gang.

168. In the comprehensive tables of cofferdam types, Mr Packshaw had included "precast concrete" under the heading "other types". Would he consider the "cofferdam" shown in the accompanying photograph to fall into this category? This last

ditch method of river closure was used during the construction of the Chute à Diable Development on the Peribonka River (Fig. 54). The blocks, which weighed some 2600 tons apiece, were slid into the river by blasting their foundations. The resulting constriction enabled the crib cofferdam to be completed upstream where previously the depth and velocity of the water had foiled all attempts to place the last cribs.

Mr Robert F. Legget (Director, Division of Building Research, National Research Council, Ottawa) wrote that the encyclopaedic character of Mr Packshaw's Paper made it at once a most valuable summary of a vitally important part of the practice of civil engineering construction, and yet one that could not well be discussed in the usual sense. Mr Packshaw's eminently practical review of cofferdam types and design was really not open to question; it was complete and was clearly based upon the careful assessment of a vast amount of broad experience. Cofferdams were, however, so widespread in use, and the Institution membership was itself now so world-wide, that some comments might be offered supplementing one or two essential points made by the author, even though these would inevitably reflect experience peculiar to the writer's own country, in this case Canada.

170. When discussing "half-tide cofferdams" (§ 18) the author stated that "sluice gates of ample proportions . . . are then essential". The importance of such provisions for the flow of water in to and out of cofferdams in tidal waters was of such importance, even though not always apparently necessary, that the writer would like to emphasize this suggested detail of design. There was at least one large cofferdam in Canada in which provision of this kind was not made with results that might have been much more serious than, fortunately, they turned out to be.

171. The structure in question was in use today as a guard pier and support for the swing span of a 1500-ft bridge spanning a river with a mean tidal range of 20 ft. The structure consisted of the main swing span support pier and an upstream and downstream protective crib, filled with concrete up to L.W.O.S.T., the three massive structures being surrounded by and connected together by rows of steel sheet piling, supported by the necessary bracing, the piling being Larssen No. 2. The structure was 428 ft long and 44 ft wide at the central pier. It was a design which combined construction convenience and permanent requirements for the bridge structure, so that the whole large "island" was built as one unit. The three massive pier structures were first formed by cross walls of sheet steel piling, of the same type as that used for the outer wall, subdividing the whole "island" into five compartments. When the last pile came to be placed and driven for the central compartment (in which the swing span supporting pier was to be constructed), the pile was not placed at low water, as had been planned, but instead was placed and driven near high tide. Steel sheet piles previously driven had been bolted to the 12-in. by 12-in. timber walers with 1-in. bolts, fitted with large cast-iron washers bearing on the timber. As the tide fell, the water level within the cofferdam remained sensibly constant, to the amazement of those then on the job who had expected the sheet piling to "leak like a sieve". Before arrangements could be made to get pumps at work, the stresses set up in the walers by the water pressure inside was such as to pull some of the cast-iron washers entirely through the 12 by 12 timbers, the sound of each one coming through being as loud as a gun shot, even as heard on shore almost half a mile away. Emergency action limited the extent of the damage; a few bent piles were the only permanent result and these were of no consequence to the stability of the structure. But if a sluice gate had been installed, much trouble and alarm could have been avoided.

172. The author naturally included timber cribs among the main types of cofferdams. Fig. 5 showed such a structure built of squared timbers and sheeted with steel sheet piling. In countries such as Canada, where timber could be economically used on construction in a way not possible in Great Britain, timber crib cofferdams were very frequently used. When constructed on the more isolated type of big job, such as water power plant projects in the more remote parts of the country, it was commonplace to secure the necessary

timber by cutting in the forests around, or by local purchase, rather than use the much more expensive squared timbers (which were now available only from British Columbia). With good woodsmen available, it was quite remarkable to see what rigid and uniform looking cribs, even of large size, could be framed from newly cut logs that had been merely barked. Correspondingly, the use of "Wakefield" timber sheeting (a simple type of tongue and groove design, obtained by the spiking of three 2-in. planks together) was much more common practice in Canada than the use of steel piling, despite the many advantages of the latter. The writer had described a major cofferdam installation of this type to the Institution in one of the "Selected Engineering Papers" that were in use about 1930[50]. The main cribs ranged up to 30 ft in depth; most of them were built upside down, cantilevered out over the edge of a crib already in place, and launched by tipping when supporting cables were cut. In this way, the expert French Canadian "river men" who built them were able to finish off the bottom of each crib to the exact contours of the river bed where the crib was to sit, previously determined by careful sounding.

173. Finally, and since Mr Packshaw's paper would almost certainly stand for a long time as the authoritative review of cofferdam design and construction, it might be of interest for younger members of the Institution to realize that cofferdams were not the "new" invention that was sometimes suggested although not in this Paper, limitations of space—it may be assumed—having kept the author from referring to the historical aspect of his large subject. There would be found in Chapter XII of the Fifth book on Architecture of Vitruvius (an architect and engineer who lived during the 1st century B.C.), some quite vivid descriptions of the construction of cofferdams, the word being used in exactly the same sense as in this Paper. Typical were the following: "Then, in the place previously determined, a cofferdam, with its sides formed of oaken stakes with ties between them, is to be driven down into the water and firmly propped there; then, the lower surface inside, under the water, must be levelled off and dredged, working from beams laid across; and, finally, concrete from the mortar trough—the stuff having been mixed as prescribed above—must be heaped up until the empty space which was within the cofferdam is filled up by the wall". . . . and (later in the same chapter) . . . "A cofferdam with double sides, composed of charred stakes fastened together with ties, should be constructed in the appointed place, and clay in wicker baskets made of swamp rushes should be packed in among the props. After this has been well packed down and filled in as closely as possible, set up your water-screws, wheels, and drums, and let the space now bounded by the enclosure be emptied and dried."[51]

Mr Laurits Bjerrum and Mr Ove Eide wrote that in § 40, p. 387, Mr Packshaw stated that in excavations in soft and medium stiff clays there was a critical depth equal to $4c/\gamma$ below which the active earth pressure computed by the classical earth-pressure theory exceeded the passive pressure. Below this depth there should thus be a net active earth pressure on the sheet pile which could only be carried by bending in the sheet pile.

175. The writers were of the opinion that this was not necessarily the case for excavations between flexible sheet piles. An investigation of a number of deep excavations carried out in Norway had shown that it was possible to reach greater depths than $4c/\gamma$ even when the sheet piles did not penetrate below excavation level. These experiences had been collected and published by the writers[30]. In this paper it was proposed that the critical excavation depth should be computed as $N_c c/\gamma$, where N_c varied from 5·5 to 9·0 depending on the shape and the depth as stated in the paper at the end of p. 40.

176. This rule was now generally applied in Norway and it was the practice that the safety factor against a bottom heave should be 1·3. A few major excavations had also been carried out, however, with smaller safety factors. As an example, the excavation for a section of the subway in Oslo with a width of 10 metres and a depth of 10 metres was carried down to a depth which was equal to $4{\cdot}8 \times c/\gamma$. In such cases the deflexion

of the sheet pile had been of the order of 4 to 8 in., which of course was taken into consideration, but no failure had taken place.

The Author, in reply, was glad to have the support of Mr Harding in favour of concreting the base of the structure under water wherever there was any serious risk of failure during the last few feet of excavation and dewatering. Mr Harding also drew attention to the possibility of piles coming out of their clutches when driving into compact gravel or cobbles. This was, however, a relatively rare occurrence with proper driving methods and also if a sufficiently heavy section of piling, preferably in a higher grade of steel, were chosen.

178. Mr Collingridge gave an instructive description of dewatering by pumping from deep boreholes in the construction of a cofferdam in very difficult ground. He referred to the discussion that preceded the decision to take the piles down into the underlying gravel instead of stopping them in the clay some distance below excavation level. This was certainly a wise precaution and the extra cost must have been trifling in comparison with that of the whole cofferdam. Another consideration was that if the piles had not been taken down into firm ground there could have been a greater risk of inward deflexion. Mr Collingridge also mentioned two cofferdams supported by berms, as illustrated in Fig. 9 of the Paper, constructed in very similar ground conditions and only 200 yards apart, and yet very different in their behaviour, especially in regard to dewatering. Perhaps this was a useful reminder that no two cofferdams should be expected to be alike.

179. Mr Kipps had given an interesting description of the use of bentonite suspensions for supporting the sides of trenches. This was a new technique and the mechanics of the interaction between the bentonite and the soil were not clearly understood. However, it was a technique likely to find wider application and it was also being used increasingly for supporting boreholes for large-diameter bored piles, thus eliminating the necessity for casings.

180. Mr Andrews had given examples of the use of steel sheet piling for permanent works in which complete watertightness was essential. In a cofferdam a small amount of leakage through the interlocks of the piles was regarded as acceptable and normal but it was not generally known that absolute watertightness could be obtained at little cost by caulking with lead wool or asbestos cement. Sheet piling could then form the walls of a structure such as an underground pumping chamber, instead of building the walls of reinforced concrete and merely using the piling as a temporary cofferdam.

181. Dr Ward had referred to the effect of frost, which could cause a substantial increase in the loads on a cofferdam. He also commented, quite correctly, that the Author's remarks about eliminating ground movements outside a cofferdam by preloading the struts could not be expected also to eliminate the vertical displacements.

182. Mr Edwards had given an example of a typical base failure in soft clay in accordance with § 40 and 68 (f) of the Paper. There were, regrettably, many case histories on record of cofferdams that had failed through the unsuspected presence of weak material that had not been discovered by the preliminary borings.

183. The cofferdam described by Mr Wakeling and Mr Hamilton illustrated both the limitations of relief wells (which nevertheless had acted as useful warning signals) and the effect of pumping from deep boreholes. However the construction of the foundation would have been very costly and very difficult if the base plug of concrete had not been formed under water. The welding of the struts and walings to the sheeting to provide monolithic rigidity to the whole cofferdam was an inexpensive but doubtless valuable precaution.

184. Mr Mackintosh had commented on certain aspects of several cellular cofferdams constructed in the U.S.A. Engineers in Great Britain have little experience of this type of cofferdam, largely because a straight-web section of piling was not produced in this country until quite recently. The Author had shown the two most common forms of cellular cofferdams in Fig. 8, the design with circular cells being the one most frequently

adopted. However, very high and consequently very wide cellular cofferdams with a correspondingly large radius for the cells or arcs could develop very high circumferential tensions. When these tensions were beyond the capacity of the piling sections, even if rolled from higher grade steel, the cells could be built in the shape of a four-leaf clover as illustrated by Mr Mackintosh in Fig. 45. These had arcs with a relatively small radius. Relief of pressure could also be obtained by a berm on the downstream side, as in that illustration and in § 59 of the Paper. The provision of a template (Fig. 51) should be regarded as essential in the construction of a cellular cofferdam.

185. In reply to Mr Ellis, the Author agreed that the double-wall cofferdam at Carrington looked very narrow. He did not know the length of the piles but did not think it would be economical or good practice to design the two lines of piling as twin cantilever walls. Moreover, cantilever walls were subject to very high bending moments as indicated by Fig. 17a in the Paper. The ratio for width to height suggested in § 52 was empirical but on the other hand it had no relation to the penetration of the piling, provided the conditions in the three preceding paragraphs were satisfied. The Code of Practice for Foundations in its early edition (now being revised) made no mention of cellular cofferdams and referred only to the straight-sided double-wall type. It discouraged the use of the latter on hard rock because of the uncertainty of securing the bottom of the sheet piling against lateral movement. The Author entirely agreed with Mr Ellis about the expense and waste of trying to drive sheet piling into rock. Fortunately that was not necessary for cellular cofferdams.

186. The double-wall cofferdam illustrated by Mr Ellis in Figs 46 and 52 showed that this type could be successful even under adverse conditions, though the Author thought that at least the silty clay and preferably all the clay should have been excavated from within the dam before depositing the rock fill. However, there were several factors which helped the stability of the dam: there were two sets of tie-rods, the lower sufficiently far down to effect a substantial reduction in the active pressure at the foot; the piling might, in fact, have been driven to obtain some foothold in the rock; the shape of the cofferdam in plan enabled it to obtain considerable support from the two abutments; and the concrete slab, even though constructed after dewatering, doubtless also helped.

187. Mr Rennie was quite justified in criticizing the Author for the limitations of the Paper and for confining his attention largely to modern practice. The Author could only plead that he, in turn, was limited in the length of a paper acceptable to the Institution. A study of Mr Rennie's Paper[36] (pp. 318–326 and Figs 4 and 5, Plate 6) would go a long way towards rectifying the omissions. Several types of cofferdams were described in that Paper: timber cribs with stone fill; rock and earth fill embankments with a steel pile core; and timber A-frame cofferdams with the upstream face formed of timber planking sloping at about 50° to the horizontal. All the cofferdams were designed to be overtopped by sudden floods, the surface of the earth fill being covered with brushwood rubble or stone pitching for that purpose. However, the Author thought that timber A-frame cofferdams were now less attractive than at the time the Chenderoh power station was built because a substantial proportion of the total cost must have been in skilled and semi-skilled labour. Under modern conditions, high labour costs were likely to force the contractor into greater expenditure on materials and equipment if this led to savings in labour charges, and this would influence the type of cofferdam that he would choose.

188. Dr Bishop had made a valuable contribution which served to emphasize that the commonly accepted width to height ratio of about 0·8 was largely empirical. However, the Author could not agree with the suggestion that it might be inadequate. It represented common practice and as far as he knew there had been no failures of double-wall cofferdams which could be ascribed to inadequate width. The one mentioned by Dr Bishop[49] concerned a cellular cofferdam in which the junction pile of one of the cells failed structurally. Moreover some theories had been published, notably by Professor

Terzaghi[31] and Mr Cummings[32], though the latter was several years after the investigation by Professor Skempton and Dr Bishop.

189. The cofferdam at Gallions lock had some features which were unusual for this type. There were berms on both sides and the inner was so large that it really made the structure into an embankment dam with a steel pile core. Another factor was in the values of the active and passive pressures due to the sloping berms, which could not be calculated with any precision in either magnitude or position. Without extending his comments to excessive length, the Author assumed that Dr Bishop had made full allowance for the vertical shear between the soil elements. This could be of great significance. In a normal double-wall cofferdam one of the main reasons for keeping down the internal water level by drain holes was to increase the shear strength of the filling as much as possible.

190. All double-wall cofferdams tended to develop substantial inward deflexions. In the case of Gallions lock this might have been aggravated by two factors: the hard-core filling, deposited through water, might have been rather loose; and the rather compressible chalk on which it was built might have permitted excessive deflexion, due to tilting settlement resulting from the consolidation of the chalk.

191. Mr Cornfield's contribution provided an instructive summary of the present state of knowledge of the design of single-skin cofferdams. His remarks showed that this could not be regarded as adequate, especially in regard to condition (4) which covered stiff clays such as London blue clay. The Code for Earth Retaining Structures[19] recommended that the forces should be calculated as if the soil were an imaginary fluid with a density of 30 lb/cu. ft, but this was merely an attempt to give some kind of guidance to the designer and, in effect, an admission that very little was known on the subject. In the Author's opinion this recommendation led to excessively high values as shown, for example, by the behaviour of the cofferdams reported in § 27. Clearly there was need for research on the lines already carried out fairly extensively for soft clays to avoid the continuation of costly over-designing.

192. The Author regretted that he had been unable to give more detailed case histories in the manner suggested by Mr Russ. He hoped Mr Russ would understand his difficulty in obtaining all the information that could be regarded as desirable, especially as it is often non-existent. In reply to Mr Russ's question about the pumping capacity required for dewatering a cofferdam in a constant water level, the Author believed that this was dependent mainly on the wetted area of the piling, i.e. the perimeter multiplied by the depth, and not on the depth. Obviously the amount of pumping alone would vary considerably from one site to another but assuming that the base of the cofferdam was impermeable, the seepage through the interlocks was likely to be roughly $\frac{1}{2}$ to 1 gallon per minute per square foot of wetted area. This was for the early stages of dewatering, until a sufficient difference in water levels had been established and the clearance in the interlocks largely sealed by the external water pressure. After complete dewatering, a pumping capacity of only about one-tenth of the figure quoted above should be sufficient to keep the cofferdam dry.

193. Mr Russ's examples of crib-fill cofferdams, taken from Canadian practice, would be of interest to engineers in Great Britain who were less familiar with cheap timber and rocky river beds. The idea of building the crib upside-down, so that the tops could be more easily tailored to the contours of the river bed, seemed particularly ingenious. As regards the method used for closing the cofferdam in Fig. 54, the Author was not sure whether this was an instance of heroism born of desperation, but he would not attempt to classify it in the table of cofferdam types!

194. Mr Legget's description of the cofferdam that tried to hold water instead of keeping it out served to emphasize the value of sluice gates, even though the Author had previously thought of them as an insurance against floods or failures and not against inexperience. With his knowledge of Canadian practice, Mr Legget had also made some interesting observations on timber crib cofferdams. His remarks were a reminder of the extent to which timber had now become a relatively uncommon material of con-

struction in Great Britain, and the use of timber almost a lost art. As in most other arts, there had been more technological advance in the last 50 years than in the preceding 2000 that had elapsed since the days of Vitruvius.

195. The comments by Mr Bjerrum and Mr Eide gave the Author the opportunity of correcting an error in the penultimate line of § 40. The expression should, of course, read $N_c c/\gamma$. In this paragraph the Author had tried to distinguish very briefly between two modes of failure. The first was concerned with a depth greater than $4c/\gamma$, which meant that theoretically no passive resistance could develop; the second referred to base heave, as described by Mr Bjerrum and Mr Eide in their Paper[30]. It was now useful to know that in accordance with Norwegian experience a depth of $4c/\gamma$ could be exceeded and that, provided a substantial inward deflexion could be tolerated, a coefficient of 4:8 instead of 4 would still provide an adequate factor of safety.

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