Paper No. 6164

**Construction of circulating-water pump-house at
Cowes Generating Station, Isle of Wight†**

by

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Discussion

Mr J. P. M. Pannell said that the geology of the Isle of Wight was almost a museum piece, and any foundation there was bound to offer a problem. The limestone beds in the island were thin, non-homogeneous, and badly shattered, but in spite of this useful supplies of aggregates and road-making material had for several years been quarried at selected sites.

56. The Cowes Harbour Commissioners had a steel sheet-piling training wall at East Cowes, which they had decided to cover in about 4 years ago and form a training bank. The training wall had been in copper steel, which had a forecast life of about 80 years, but after 18 years the Commissioners had found that it was necessary to strengthen it. It was at first proposed to use the local limestone, since local opinion had been that the limestone would be adequate for tipping and forming a bank to enclose the steel sheet-pile training wall. In the event, after testing at three quarries, its size had been found to be hopelessly small owing to the shattering. The largest pieces available had been found to be about 1 cu. ft. Therefore the embankment project had been dropped, and the steel sheet-pile training wall had been backed with concrete.

57. The same shattering of the limestone appeared to be the basic cause of the trouble referred to in the Paper. It was fortunate that the outcrop referred to by the Authors had been so near the site of the work and also that the fissuring in the clay had occurred, because otherwise a very heavy artesian pressure could have arisen because of the rapid drop of the limestone stratum, which was almost vertical where it outcropped along the high ground of the island, and which then dropped down sharply in a distance of 3 or 4 miles to the site of the work described.

58. The references in the Paper to grouting by the Colcrete process afforded another instance where colloidal grouting technique had offered a valuable practical solution in under-water concreting. The order of operations given in § 35 did not make it clear if colloidal grouting had been used for operation (5) and also for grouting up the cables, i.e. the grouting before placing the large rejects and grouting up by the Colcrete process.

59. Finally, in view of the failure in one case of the single wire of the cable and in another case of the complete cable, had the Authors any views on a modified bottom anchorage or the possibility, even though it would involve sharp reversed bend, of inserting the wires as pairs with the bottom loop?

Mr. Savile Packshaw observed that the manner in which the cofferdam had failed and the subsequent thorough investigation of the ground showed that the failure had been

† Proc. Instn civ. Engrs, vol. 9, p. 217 (Mar. 1958).

due, probably wholly, to the upward forces within the clay caused by the artesian water pressure. It was significant that the failure had started in the corner of the cofferdam where the sand stratum was at its highest. The slowness of the failure also suggested that the artesian water pressure was to blame and the fissures subsequently discovered in the clay would seriously aggravate that condition. Moreover, the strength of the clay was not so low as to indicate that it could not develop the passive resistance required to ensure the stability of the lower part of the sheet-piling.

61. The vane tests mentioned in § 9 had been made at another part of the site, and it was unlikely that the shear strength of more than 2,000 lb/sq. ft obtained from the vane tests did in fact exist in the neighbourhood of the cofferdam. Nevertheless, Mr Packshaw believed that some tests made after the failure showed that the shear strength varied from about 600 lb/sq. ft near excavation level to about 1,500 lb/sq. ft lower down, which should have ensured that the lower part of the piling did not move inwards as it had done.

62. Was it really essential to anchor the base of the slab to the underlying rock? Mr Packshaw emphasized that the cofferdam had failed only when excavation was practically complete. Therefore, a factor of safety of 1 had almost been achieved, and he thought it followed that the upward forces due to the artesian water pressure had been almost balanced by the resisting forces. After the concrete slab had been placed under water, those forces were, first, the adhesion between the concrete and the surrounding sheet-piling, and secondly, the tensile load that could be resisted by the box-piles even without anchoring them to the rock.

63. In § 31 the Authors stated that the upward force at excavation level had been calculated to be 900 tons based on an artesian water level of - 6. He had been unable to calculate the force as more than 700 tons.

64. Mr Packshaw suggested that one procedure would have been to drive the box-piles under water, and then to deposit a 3-ft slab of concrete under water (as described in the Paper) and connect the slab to the box-piles. According to his calculation, the dead weight of the slab was of the order of 300 tons, which left about 400 tons to be carried by the adhesion of the concrete to the steel or by the anchorage of the slab to the box-piles, or by both acting together. If the whole of the uplift was resisted by the slab by itself, the adhesion strength between the concrete and the sheet-piling would amount to only about 10 lb/sq. in. A more positive connexion which would make it unnecessary to rely on adhesion could be also obtained by some fairly simple work by divers. However, since the slab was not heavily reinforced, it might not be able to span the cofferdam, and in practice it would therefore be necessary to bring the box-piles into action.

65. Referring to the loading of the box-piles, Mr Packshaw said that the primary purpose of those piles was to carry the weight of the finished structure, and each was then loaded to 45 tons. He thought that 45 tons on B.P.3 box-piles driven down to rock was very moderate, and in his view the piles could be limited to, say, about twenty-five, and loaded to 80 tons each.

66. Reverting to the net uplift figure of 400 tons which he had mentioned, even if there was no adhesion at all between the concrete slab and the sheeting of the cofferdam, so that the whole of that force was carried by the tension in the box-piles, it would amount to only 16 tons/pile on the reduced number which he had suggested. This corresponded to an adhesion of about 300 lb/sq. ft on each box-pile, and the piles should have carried that load safely. If there were forty-five box-piles, as had been actually used, then of course the tension in each would be smaller—about 11 tons.

67. Another possible way of neutralizing or reducing artesian water pressure was by the use of relief filters. He was referring to a method described by Ward¹ in *Géotechnique* in September 1957, in which the case history was similar to that now under discussion. The method had been to relieve artesian water pressure in a stratum of laminated clay by putting down perforated tubes filled with gravel to act as relief filters. Only a small quantity of water issued from the filters, and it had simply been allowed to flow away to a sump from which it was pumped out. There had been no

pumping from the filters themselves. Could the same method have been adopted at Cowes? In that case, none of the measures that had been actually adopted, or the alternative procedure which he had outlined, would have been necessary, and the de-watering of the cofferdam would have proceeded in the normal manner after the installation of the filters.

68. Finally, how much had it cost to form the anchorages, including the cleaning out of the box-piles, drilling the holes in the rock, and providing and installing the cables?

Mr E. M. Hayward observed that the cause of the failure had been clearly stated to be the unsuspected artesian head of water in the layer of sand. Although the clay during excavation had been sufficiently hard to warrant the use of compressed-air clay spades, the water had considerably softened it and its cohesion had been gradually destroyed by the opening of the fissures and the entrance of the water. That clay was almost like the London blue clay—very firm and hard.

70. Referring to § 28(c), he suggested that the inward yielding of the steel sheet-piles was not therefore a cause or even a contributory cause, but a consequence of the failure of the clay and the destruction of its passive resistance. There had been a gradual inward yielding of the 45-ft Larssen No. 2 piles. This was not uncommon in soft ground, and should be allowed for in the design of wall thicknesses and reinforcement cover. This inward movement of the piles had not given any particular cause for anxiety, until, after the bottom had started to come up, there had been a deflexion at the bottom of about 12 in. on the sheet-piles. It was evident now that it would have been better to have used Larssen No. 3 piles, which would have been stiffer and would have prevented that gradual inward deflexion.

71. Originally they had intended to use five timber frames. The fifth frame had been omitted and reliance placed on the passive resistance of the alleged stiff clay. The upper two frames had been placed from mud level, in the dry, from low-tide level, and the lower two frames had been floated in after most of the cofferdam had been constructed. Any more frames would have caused congestion and difficulty in placing the reinforcement and the box-piles which had to be driven afterwards. For these reasons also, the layout of the frames was unusual. It was obvious, however, that a fifth frame could not have prevented the upward bursting, as was perhaps suggested in § 28(c).

72. Although the clay appeared to be stiff, in fact shear vane tests made after the failure had shown that the strength of the clay was in the region of 150 to (at the most) 600 lb/sq. ft, in other words, the equivalent of very soft clay. It was difficult to reconcile those very low figures with the apparent firmness of the clay during excavation and the high figures, in the region of 2,000 lb/sq. ft given in the Paper. However, this did act as a warning to designers of retaining walls, cofferdams, and such structures of the very rapid softening of the clay which could occur whenever it was disturbed and water was allowed to enter the fissures.

73. He thought that his figures might show that sole reliance on the adhesion of box-piles without any sort of prestressing, as suggested by Mr Packshaw, would have been rather risky, and by then those concerned had had quite enough trouble!

Mr J. A. Williams referred to § 33, where it was stated that a calculation had been made to check the stability of the structure against sliding. Had the effect of the piles been omitted from that calculation and the figures taken out merely on the mass of the structure?

75. According to § 35 (1), the box-piles had been driven to what appeared to be a very stiff set of 50 blows per inch. Had the contractors and the engineers been satisfied that they were getting penetration all the time and had there been much deformation of the pile heads?

76. Reference was made in § 35 (5) to the 6-in. layer of grout placed under water. The chances of this being effective were not very strong. As grout, it was not of

much importance, but Fig. 7(c) showed the reinforcement placed immediately on this layer, and he imagined that the steel would, in fact, not be protected on the under-side.

77. The main point to which he wished to refer was the anchorage of the foundations, all of which centered around the prestressing of the box-piles. Mr Williams had become convinced that this had been unnecessary, and that the box-piles themselves would have been adequate, acting during the construction stage as tension piles and in the permanent stage as bearing piles. His own calculations confirmed the figure of 900 tons uplift, shared by forty-five piles; allowing 250 tons dead load for the floor slab, this resulted in 14 tons tension per pile. If it took fifty blows to the inch to get the piles in, it would take much more than 14 tons to pull them out. However, he agreed with Mr Hayward that a stage had been reached on the site when no more chances could have been contemplated.

78. Therefore referring to § 35, his own suggested procedure was as follows:

- (i) Drive the forty-five piles and then put a pull test on half a dozen of them; this could probably be done in a week.
- (ii) If the pull tests were satisfactory, then proceed to dewater and finish the job without further trouble.
- (iii) If the tests raised any doubts, then either (a) add more concrete under water; or (b) if the levels did not permit this, construct the walls under water until there was enough weight; or (c) if that was inconvenient, then use temporary kentledge on the bottom.

Any one of those alternatives would have been sound engineering and would have saved both time and money.

79. If the Authors were faced with the same problem again, would they give more serious consideration to this alternative solution, which was one that had apparently been dismissed rather summarily in the stress of circumstances.

Mr Stanley Rodin said that the Paper drew attention to the serious limitations of probings as means of determining the soil conditions at a site. As often happened in practice, it had been found that probings were not a satisfactory substitute for borings. In particular, they gave no information on ground-water conditions, which were vital to the case under discussion.

81. In the initial site investigation, the six boreholes had provided apparently adequate information of the ground conditions at the site of the generating-station extension. However, in view of the location of the pump-house, it was unwise to have relied on two probings in the pump-house site where the soil and ground-water conditions were obviously an important factor in the design of the structure and the method of construction. Even one borehole would have shown that the soil conditions were very different from those on the extension site.

82. Subsequent to the failure of the cofferdam, Mr Rodin's firm had been called in to investigate the soil conditions on the pump-house site and ascertain the cause of the failure. The results of that investigation were summarized in §§ 23–29. Referring to Figs 5 and 6, the ground water conditions in the sand layer were of particular interest. In some of the boreholes stand-pipes had been inserted in the sand and carefully sealed off from the overlying strata. Readings over a period had shown that the ground-water in the sand was under a fairly high artesian pressure. At the time of the investigation, the tidal fluctuation had been approximately 7 ft, whereas the fluctuation observed in the stand-pipes had been generally from 2 to 3 ft, and the highest water level in the stand-pipes had been 1 or 2 ft below the low-tide water level in the cofferdam. Also, the chloride content of the ground-water samples taken in the sand had been very low compared with the normal chloride content of the river-water. Those observations had indicated that the ground-water in this layer had not been connected to the river at a source close to the pump-house.

83. In connexion with the anchorage system used in the remedial measures, what criterion had the Authors used for assessing the pull-out resistance of each anchorage?

Had they assumed that the resistance around each anchor was given by the submerged weight of an inverted cone of rock, or had they assumed that the anchors were too close together for them to act independently?

84. He also noted that each wire in the cable anchorage had been stressed separately. Would it not have been preferable to have stressed the complete cable as a unit in order to ensure that the bond value of the anchor cable in the concrete was capable of withstanding the total design pull-out load?

85. In § 46 the Authors had mentioned that they had measured the deflexion of the piles by an engineer's level. He thought that this would give only a rough guide of the load in the pile, particularly since the deflexion was only about $\frac{1}{8}$ in. The method of stressing the anchor by using the box-pile as a reaction might lead to a misleading impression of the pull-out resistance of the anchorage. Referring to Fig. 7c, if a pull was applied to the cable an equal downward load was applied to the pile. This load compressed the rock around the anchorage, which would tend to give a higher value of the pull-out resistance than would occur in practice, since the uplift loading on the structure would in fact cause an upward pull on the piles.

Mr A. E. Johnson corroborated Mr Pannell's remarks regarding the haphazard nature of the limestone strata beneath the site. The variations of those strata were very large in comparison with the size of the site and had made it difficult to decide what lengths of box-piles to use.

87. In order to keep a check on the position of the toes of the piles in relation to the limestone the toe levels were plotted against data obtained from the boreholes. In the most northerly row of piles (Fig. 4) all piles had penetrated the limestone up to depths of 10 ft. A set had been specified and agreed with the contractor to avoid shattering the limestone strata. Piles Nos 6, 14, and 18 did not achieve a satisfactory set at the required level, replacements were driven 4 ft 6 in. away (piles Nos 6A, 14A, and 18A). The latter two piles finished with their toes 10 ft above the level of piles Nos 14 and 18.

88. The piles in the most southerly row (Nos 37-45) finished above the predicted level. After coring, prior to drilling the anchorages, with the exception of pile No. 44, the piles were resting on the rock. It was found that pile No. 44 was founded in the sand layer with the toe 2 ft 6 in. above this rock, but despite this satisfactory anchorage and stressing of the cable had been achieved.

89. Finally, with reference to the diving operations, Mr Johnson stressed that if a specification called for a diving suit to be provided on site, then whoever used that diving suit should be carefully supervised, should have undergone proper training, and should be medically fit.

Mr M. J. Tomlinson felt that there was one important feature of the design of the cofferdam which had not been mentioned in the Paper and which had not been referred to by previous speakers. He referred to the fact that there were no external ties or frames provided around the sheet-piling. He thought that such a frame was regarded as more or less essential for any cofferdam constructed in tidal water, and such a frame would have provided a restraint at the top level which, if not entirely eliminating the inward yielding, might have cut it down to a considerable degree. It was possible, that if external top and lower frames had been provided, the failure might not have occurred.

Mr H. J. B. Harding agreed with what Mr Rodin had said on the subject of probing. Sometimes both probing enthusiasts and geophysicists claimed too much, and this had been guarded against in the Code of Practice on Site Investigations.

92. Mr Harding said that one of the valuable lessons to be learnt from the Paper was the need to look for the possibility of artesian water under pressure when designing such foundations as described in the Paper.

93. He thought it might be useful to compare the findings in this Paper with one or two lessons from the past. The best-known one concerned the George V Graving Dock at Southampton.² There Mr Wentworth Shields had put down additional

borings and found green sand with an artesian head higher than the ground level, at a depth of 200 ft. By this foresight he had been able to avoid the risk of a burst from below when his excavation reached 100 ft, and this had been the first case when deep wells for water lowering had been used in the United Kingdom.

94. Soon afterwards there had been another case where a failure had actually occurred, because of artesian water. At the entrance to Grimsby Fish Dock excavation had been carried out in soft clay inside a cofferdam, and piles had been driven into gravel and sand below, when the artesian head in this sand had burst up through the soft clay, and 70 cu. yds of sand were brought into the excavation, and some of the completed concrete wall had had to be rebuilt. Here again, six deep wells were installed to remove the water head, but after the damage had occurred.

95. In the case described in the Paper, a test well had been sunk and the yield was thought to be too small to be helpful, and probably the solution they had adopted was the most reliable in the circumstances.

96. However, a number of wells produced a much greater general lowering than a single well, and it should be remembered that in some cases it was the head of water and not the volume which was the cause of danger, and also that the greater the area to be covered by wells, the more satisfactory the result.

97. Another similar problem had arisen at Shellhaven, described by Skempton³ in *Geotechnique*, where a large pump-house had been constructed inside a cofferdam in very soft estuarine clay, and the bottom of the pump-house founded on piles driven into Thames gravel beneath, which was under an artesian head. In this case deep wells formed part of the design, and the artesian head was reduced before excavation commenced. At the time there were some who criticized this as excessive caution, but the behaviour of a shaft in the cooling-water tunnel which was sunk nearby, proved that the precautions had been necessary.

98. The failure of the cofferdam at Battersea Power Station⁴ resembled the occurrence at Cowes, but here the steel piles had been driven through Thames ballast into London Clay, and the failure may have been due to insufficient strength of timbering at the lower depth.

99. In the case of the pump-house at Fawley, the problem again had been to build a pump-house in very soft material, with hard ground 30 ft or so below the floor of the pump-house. In this case instead of driving concrete piles inside a steel cofferdam, the cofferdam itself had been driven with long steel piles penetrating into the stiff Barton Clay, and excavation of the soft clay carried out under water. Plate girders had been placed between every other pile for about 30 ft or so above the stiff clay, and secured by previously welded cleats, and the whole floor of the pump-house concreted under water with bottom-dumping skips, and the cofferdam then pumped out successfully.

100. In the case described in the Paper, the under-water floor was produced by "colloidal" concrete, in other words, grouting an aggregate by means of a series of pipes controlled from the surface.

101. Could the Authors make some comparison between the cost of concreting under water and grouting an aggregate with colloidal concrete? The cost of the materials would be much the same in each case, but the handling of the grouting pipes and the pumps, and controlling a uniform delivery of the grout, employed quite a large gang of men, and some comparison between these two methods would be of interest.

Mr C. M. Strachan observed that the costs given in § 53 indicated that the work had cost 75% more than had originally been intended. The excavation depth for the substructure had been fixed at -21.75 O.D. and its finished level at +9.00 O.D., so the volume of the substructure appeared to be $45 \times 35 \times 30.75 = 48,400$ cu. ft and the extra cost at 10s/cu. ft would work out to £24,200. Could the Author say who paid for this extra?

103. Paragraph 11 of the General Conditions of Contract, 4th edition, 1955, required that the contractor shall "... inspect and examine the site and its surroundings and shall satisfy himself before submitting his tender as to the nature of the ground and

subsoil (so far as is practicable . . . etc.)” Would the Author say if the physical conditions encountered had been regarded as something which “could not be reasonably foreseen by an experienced contractor” and if paragraph 12 (2) of the General Conditions of Contract then became operative.

Mr Coates, in reply, said that the Authors were very grateful to all those who had taken part in the discussion, in which the views expressed and the points developed had helped to clarify a rather complex situation in which the Authors themselves were not prepared to be dogmatic.

105. **Mr Tomlinson** in § 90 and **Mr Hayward** in § 71 had both referred to the effect of wooden strutting. He agreed with **Mr. Hayward** that the size of the piling and the number of wooden struttings had not caused the failure, but when the failure occurred they had probably allowed greater movement than might have taken place. An external top frame had been provided by the contractor, and both tie rods and wires had been used to restrain the top of the dam.

106. **Mr Coates** had been particularly interested in the doubt (expressed by **Mr Packshaw** in § 66 and **Mr Williams** in § 67) as to the need to tie the concrete slab to the rock base at all. He was also interested in the associated question (raised by **Mr Rodin** in § 85) as to what actual functions the stressed wires performed—if they did in fact tie the slab to the rock, and provide an overall downward force to prevent further breakthrough, or if they only compressed the box-piles and did nothing else.

107. No exact and proven decision could be arrived at on these points but the logical steps controlling the decisions made could be looked at. The contributors to the discussion themselves appeared uncertain as to the condition of clay remaining over the water-bearing sand. It had undoubtedly originally had ample strength theoretically to prevent the original failure although it had not prevented this failure; but the clay had subsequently been disturbed, had been flooded for 12 months and the box-piles were driven from the overlying water through the clay to the underlying water-bearing strata. The Authors had carried out elsewhere, in connexion with their overhead line construction, considerable investigations on the strength of piles to withstand uplift, and in their opinion, irrespective of the difficulty of driving the piles, they considered it would have been imprudent to have relied on the uplift strength of the piles as driven.

108. At best, the wires achieved a connexion with the underlying rock of a known pull-out strength holding the whole structure to the ground until its weight produced stability. At the worst, the tension achieved in the wires merely compressed the box-piles, but even under these conditions, since they knew that the wires were held at the bottom, they did know that the uplift strength of the piles was greatly augmented by the grouted limestone mass at the foot of the piles.

109. The calculation of the net upward force on the concrete slab had contained uncertainties; mainly the weight of the underlying clay, and how much of this weight was by-passed by the water under pressure traversing fissures. The Authors now regretted that they had not put a flat jack underneath the concrete slab before casting. When the job had been de-watered, they would have been able to pump up the jack and readily find out the pressure between the clay (or the water if it was there) and the underneath of the filling slab. Short of having done that they knew no other way of finding the exact figure to take as the uplift force.

110. The other point of general interest was the possible value of earlier bore-holes on the cofferdam site, involving the questions (i) If they had made bore-holes would the artesian pressure have been found? and (ii) If it had been found, would an alternative design have been adopted?

111. With regard to the first question, the Authors doubted whether a normal bore-hole investigation into the bed of a tidal estuary would have found an artesian head of about the same head as low tide. Where an artesian head on shore was above ground level it left no doubt as to its existence, and at sea almost similarly, but in this case the flow was fairly restricted and the pressure low, and great care had to be taken to seal the actual bore-holes made when they had an idea of what they were looking for.

112. With regard to the second question, it was not possible to clear one's mind and design afresh. The strengths of the clay measured would have indicated ample strength to withstand the artesian uplift and what modifications might have taken place was a matter of pure conjecture.

113. In reply to Mr Rodin (§ 84) the stressing of each wire separately had been the custom and practice of the contractor and it had seemed a very sensible procedure.

114. Finally, the art of the civil engineer was to use his judgement based on incomplete knowledge of the whole situation. "The solution adopted had not been extravagant; it had been cautious but not unreasonably so; it had had a certain element of technical elegance, and the pump-house had been completed; in other words, it had worked.

Mr Slade said that Mr Pannell's observations on the nature of the Isle-of-Wight limestone confirmed the Authors experience at Cowes. The grout carpet laid to seal off the underwater excavation was colloidal grout placed by the main grouting pump. The same material was used to bond the cables but a smaller force pump was used. Looped cables were considered but it was thought that the larger anchorage holes which would have been required would have needlessly added to the expense of the operation for little more security of the anchorages themselves.

116. The Authors had come to the same conclusion as Mr Packshaw regarding the factor of safety, and it was considered that if the base slab had been constructed speedily in strips, after the initial excavation to formation level, then in all probability the pump-house could have been constructed as planned.

117. Detailed calculations very similar to those mentioned were made at the time of the failure, and if there had been less urgency in commissioning the generating station then possibly a less conservative design of remedial works would have been undertaken. In defence of the box-piles of B.P.3 section, great difficulty was experienced in drilling through the limestone in the additional site investigation, and it was considered essential to avoid deformation of the toes of the piles on entering the rock. Also, because of the inclination of the upper face of the rock strata, it was felt that lighter piles might have "run off" and complicated drilling for the anchorages. The use of relief filters was considered, but as little information on the subject was available at the time, doubts of their efficiency and the possible cross-section of such filters, combined with the small output of water from the pumping tests, caused this idea to be abandoned.

118. In connexion with the calculations on the stability of the structure against sliding, no account was taken of the shear value of the box piles and the moment to be resisted was calculated solely on the mass of various elements in the section. By this method there was a hidden factor of safety from the contribution of the piling.

119. No material damage to the pile heads was caused by the set of 50 blows per inch. Evidence from the drilling operations indicated that there was very little running off or distortion of the piles. The bottom steel in the underwater slab was necessary only when the slab was acting as a diaphragm against the artesian pressure and was consequently considered as temporary works. The Authors could not agree that the 6-in. grout layer was not a satisfactory bed on which to place the reinforcement. Suggested procedures outlined in § 78 were considered, and given the problem again it is possible that the Authors would be willing to undertake more underwater construction now that they had greater experience of the technique.

120. The criticism by Mr Rodin of the limited site investigation on the pump house site was accepted. In the design of the anchorages the Generating Board had relied on the experience of the specialist subcontractor who had carried out similar work on a dam. In view of the close centres of the anchorages it was reasonable to assume that the anchorages acted *en bloc*, particularly in view of the fact that the repeated grouting-up of the bores must have eventually converted the fractured limestone strata into a solid mass of grouted concrete. The use of an engineer's level to measure the deflexion of the piles under stress was not intended to be a precise measurement, but merely to give a rough check on the induced loads.

121. Mr Johnson's contribution, in describing the varying ground conditions regarding the piles, emphasized the problem which the contractor had to contend with in carrying out the remedial works. Site staff had to make day-to-day decisions regarding the extent of acceptability of piling sets and suchlike. Mr Johnson was one of the Board's engineers who undertook diving inspection in the cofferdam at Cowes and carried out a very good job under difficult conditions in the confined and congested dam.

122. The Authors acknowledged the help that the contractors' working divers gave to the Generating Board's newly trained engineers in carrying out their supervision. Fitness, and the need for regular training of civil engineering staff who are likely to be employed in such different operations, could not be overemphasized.

123. The provision of the anchorages and the underwater slabs cost £3,000. Mr Strachan's questions were very proper in respect to the management of the contract, but the Authors felt they were outside the scope of the Paper, which dealt purely with the solution to the physical problems.

124. The Authors appreciated Mr Harding's contribution to the discussion. They had no firm comparative costs for grouted concrete and the traditional concrete placed under water. The grouting technique had much to offer both under water and in the dry.

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