Paper No. 6334

Construction of a large experiment tank at Haslar †

by

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Discussion

The President said that reference had been made to discarding the plastic design method, which assumed that a succession of plastic hinges formed in a structure. Almost the only foreseeable cause of such behaviour was differential settlement resulting from unsound foundations, and one thoroughly good reason for using this design method seemed to be that if differential settlement did in fact occur the structure would be adequate to cope with the resulting conditions. The Authors had, however, stated, in § 40 that: "While a further saving of 100 tons of steel might have been possible by using plastically designed portal frames, such a design was not considered practical on foundations which would settle unevenly". This seemed curiously illogical to the President and he asked for the Authors' explanation.

Mr D. H. Little (Superintending Civil Engineer, H.M. Dockyard, Rosyth), who had been in charge of the design team, said that if plastic hinges formed in such a large span the engineer would certainly know where he was but might not like being there.

59. There had been two points of interest in the design, a very big tank and a very big roof, and the designers had decided to deal with the tank first. They had wanted to found it as low as possible, to reduce settlement, but the idea of building it like a graving dock and putting it well down into the ground had had to be discarded, and they had chosen a floor level of +5 O.D. If they had gone higher they could have avoided the water trouble, but they might have had worse settlements. In view of the rigidity of the design, it might have been better had they gone a little higher.

60. In cross-section the wall was essentially an I-beam, with the walkway as the top flange, the footing as the bottom flange, and the wall proper as the web. He had been anxious to make the whole foundation rigid, and the beams acted as horizontal girders between two buttresses, the unit, 45 ft long, being designed as one complete whole. The first proposal had had only one expansion joint in the middle of each of the 400-ft walls and no expansion joints in the cross-walls, but there were to have been sliding joints at the four corners and two in the long walls. That had been changed, because it had been decided to follow the Code of Practice and have expansion joints at not more than 130-ft centres, and so they had ended up by having ten expansion joints. What they had lost on rigidity on the overall length of each wall they had gained in rigidity at the corners. His objection to the expansion joints had been that they might be a source of weakness, but they had not been, and he thought that something had been gained by having more rigidity at the corners.

61. The big argument at the first design meeting had been on whether or not to

† Proc. Instn civ. Engrs, vol. 12, p. 57 (Jan. 1959).

have a rigid wall. The alternative would have been for the wall itself to be hinged in bitumen seated in the footing, and hinged again in bitumen seated in the top beam. That would have done away with the longitudinal rigidity, but it had been argued that one could not expect to place a wall on top of a footing which had already set without getting cracks, and that it would not be possible to place the walkway on top of the wall without getting cracks. He had held out strongly for rigidity, and had felt throughout that that should be the fundamental approach to the problem. He believed that the right decision had been taken, in view of the settlement which had taken place.

62. The cracks were unfortunate because their occurrence indicated that there had been a failure in design somewhere. The Authors had stated that water had found its way through only two places—at those cracks and at the holes formed by the shuttering tie bolts. He would never have thought that water would get through shuttering holes, and it was obvious that in future similar work they would have to be filled more thoroughly.

63. The decision to place wall concrete in one pour had not been his. He had felt that it should be poured in steps of about 3 ft, which would admittedly have given more construction joints. He would have gone from buttress to buttress in 40-ft lengths and have had 220 linear feet of construction joints per bay, whereas the method adopted, including the gaps gave approximately 120 ft. He had felt that placing in 3-ft lifts would have been preferable, on the ground that it would be difficult to pour the concrete in lifts 18 ft high. Shorter lifts too might have reduced cracking.

64. The floor, he thought, was a little elaborate, being broken up into so many panels, and he did not think that it needed any reinforcement. It had been tanked with bitumen, and the concrete was simply protection to the tanking, so that he did not think it should have been necessary to take so much care to eliminate contraction cracks. The tank, however, had proved to be 100% watertight, and apart from the vertical hair-cracks which had appeared, the design and construction had been proved. He would have liked to find the cause of the cracks, so that he would know what to do next time.

65. For the roof they had got out a plastic design, although to some extent with their tongues in their cheeks. It had been in the very early days of plastic design. They had at that time done more plastic design and construction in the Admiralty than anyone else, but their biggest span had been about 80 ft. They had gone from 30 ft to 50 ft and then to 80 ft, but to go straight from 80 ft to a span of 220 ft would have been unnecessarily bold. They had had a rigid tank design, but it had been flexible otherwise, for it had "stalks" sticking up as roof columns, and within limits it was possible to put any sort of roof on them; it could have been aluminium or concrete or an ordinary riveted steel design or, as had been chosen, a welded tube design. Their design for an orthodox riveted steel roof had been worked out in complete detail. It had probably been a little on the heavy side. The tube design had also been worked out in complete detail, and it had beaten the orthodox design, which had been fully priced, by the amount stated in the Paper.

Mr R. N. Newton (Superintendent, Admiralty Experiment Works, Haslar) said that the main building accommodated two major facilities. The first of these, the rotating arm, had been needed for very many years. Its main purpose, so far as surface ships were concerned, was to investigate directional stability, or course-keeping qualities. This had not previously been possible in existing ship tanks because of their restricted width. Although it was possible in such narrow tanks to use a small rotating arm and small models, the scale-effects on the latter were very large and this made the results unreliable.

67. So far as submarines were concerned—and that was where a great deal of the interest lay—the problem was to investigate dynamic stability, or the response of the submarine to the movement of its control surfaces; to do this it was necessary to determine what were known as the "stability derivatives". He did not propose to go into details about these, but mentioned that it was necessary to measure the forces and

couples on a model as it followed a curved path. For this case also, existing tanks were not suitable, and the rotating arm at Haslar would enable the investigations to be carried much further. In the meantime, the Admiralty had had to resort to the device of running curved models on a straight path, which had its limitations, but which had taken them a long way towards a solution.

68. Formerly, large-scale model experiments on turning (measuring the diameter of the circle in which the ship turned, the drift angle, and the speed on the turn) had been carried out in a lake on Portsea Island. A zephyr of wind provided by nature was a gale to a model, even though the latter might be as long as 20 ft, and the advantage of an enclosed waterway was obvious. The high accuracy of experiments already carried out in the tank had proved this beyond question.

69. The second major facility was the wave-making installation, by which it would be possible to make ship-motion investigations in irregular seas for the first time. There were about fifty large ship tanks in the world. Many of them could generate waves, but, generally speaking, only regular long-crested waves with a common wave-length and wave-height. Models were run in them to measure pitch, heave, and acceleration in vertical plane only. Some successful attempts had been made in Holland to go further than this, but the real need was for experiments in a truly irregular sea.

70. It was not easy to produce an irregular sea. Having produced one it was easy to recognize; and having recognized it, it was not easy to repeat it. The problems involved were difficult, but must be solved in order to compare the motion of a model in its six degrees or modes of freedom, with the motion of a ship. It would be necessary to use the most modern instrumentation, including computers, and to employ spectrum analysis. The whole process was necessarily a lengthy one, but it should be possible to see the light in 2 or 3 years' time.

71. The benefits which accrued from such research establishments in the United Kingdom were not always recorded or appreciated. From the financial point of view they paid for themselves over and over again. For instance, a tanker of 19,000 tons dead weight at 19 knots requires about 19,000 s.h.p. If such a ship ran for two-thirds of its life, taking that as 30 years, and if changes or corrections in design indicated by model experiments afforded a reduction in total resistance of only 5%—a modest figure—the saving in fuel costs alone could amount to £600,000, which was about half the cost of the tank at Haslar. Bearing in mind that millions of tons of shipping were built all over the world every year, it was obvious that model experiments were a good investment.

72. From the results of model experiments it was possible to avoid serious material failures or structural damage, and to discover weaknesses in design which might otherwise endanger the safety of the ship. Those benefits applied to any type of ship, but for warships there were other benefits to be derived, because their operational requirements were far more rigorous.

73. In conclusion, Mr Newton conveyed the appreciation of the Director-General. Ships, to the Civil Engineer-in-Chief of the Admiralty and to those of his staff who had been intimately connected with the project, and particularly to the Authors, for the prominent part they had played in bringing it to fruition.

Mark Woolfson (Senior Engineer, Ministry of Works) said that he and his colleagues had collaborated with the Authors and other members of the Civil Engineer-in-Chief's Department of the Admiralty on structural matters which concerned the rotating arm and the wave makers. The design of those facilities had made it necessary to impose certain requirements on the structural design.

75. He referred to the concrete pillar on which the rotating arm was founded and the importance attached to achieving maximum rigidity of this structure, which because of other operating requirements in the tank was limited to 20 ft dia. The model under test was attached to a travelling carriage and the arm was balanced with the carriage in the mean position. In any other position an overturning moment of up to

190 tons-ft was transferred to the foundations. The Civil Engineer-in-Chief's Department had, therefore, been asked to produce a structure of maximum rigidity under a load of 120 tons and the maximum overturning moment of 190 tons-ft. These conditions had been applied after erection of the arm was complete by loading the carriage at the mean position and moving it through a suitable distance. A deflexion of the centre post of 1½ second of arc had resulted, and since the whole of this could be accounted for within the centre post itself, it was concluded that the deflexion of the concrete structure was negligible.

76. He added that of the total overturning moment of 190 tons-ft, the greater part resulted in a static deflexion which, while undesirable, could in practice be allowed for in setting up the model. There was, however, an element of up to 25 tons-ft due to the hydrodynamic forces on the model which could not be compensated, and which was tied to a total permissible deflexion of the arm structure of 0·050 in. This figure had had to be entirely taken up in the design of the arm and left no tolerance for the foundation pillar. The test results had, therefore, been very satisfactory.

77. Mr Woolfson then showed a photograph illustrating the method of bringing the 60-ton centre-post assembly into the building and across the tank to the concrete pillar. He referred also to the reinforced concrete columns which supported the hydraulic rams operating the wave-making plungers. The rigidity of these columns had proved to be a vital factor in the function of the wave-maker control system, and tests had shown that under an overturning moment of 18 tons-ft the vertical deflexion in the line of action of the rams, 3 ft from the face of the columns, was 0·002 in. maximum, which had been considered very satisfactory.

Mr F. R. Bullen (Partner, F. R. Bullen & Partners, Consulting Engineers) asked for more information about the piles. Fig. 2 showed how confused the strata were and this led him to wonder if the upper layers represented alluvium and plateau gravel overlying the more substantial Bracklesham beds which occurred at some depth below. This confusion was well illustrated by the contrasting data in the Paper. For instance, in § 7 it was stated that when the 20-ft greenheart piles were driven, the ground heaved about 2 in. around the piles. A brief calculation showed that the extent of heave corresponded to the cubic capacity of a pile. This suggested that the pile had been driven through clay or some other material which resisted at least, temporarily, the penetration of water. Should the statement be qualified to the extent of saying that around *some* of the piles the ground heaved?

79. In § 7 it was also stated that the 20-ft greenheart piles had been driven to a driving resistance of approximately 100 tons each. A pile driven into gravel might have a bearing capacity approximating to the driving resistance, but a pile driven into clay with a driving resistance of 100 tons would certainly not have a bearing capacity of 100 tons. Furthermore, if the material at the toes of these piles was in the nature of a quicksand, then the driving resistance might have been 100 tons, but the bearing capacity would have been appreciably lower. Had any of the piles been tested by re-driving after they had stood for a time, and if so, what results had been obtained?

80. Mr Bullen said that he did not understand how these piles acted in the finished structure. It seemed from the results of test pile No. 37 that a pile carrying 100 tons would settle about ¼ in. His impression was that piles driven into this ground would act in a way comparable to the reinforcement in reinforced concrete, which depended for its action upon the fact that the concrete and steel suffered comparable strains. If, therefore, the pile settled ¼ in., would not the adjoining ground also settle ¼ in.? From Fig. 3 it was found that ¼ in. corresponded to a loading of 50 tons on a 5-ft × 5-ft plate, namely 2 tons/sq. ft. It followed that the ground, if it only settled ¼ in. would be taking 2 tons/sq. ft. Therefore, why introduce the piles? Under the test loading the settlements seemed to have been of the order of ½ in., which suggested that the piles tended to project up into the floor and therefore carried a much greater load. But if the piles were not carrying the loads, and were moving down into the ground, why have the piles?

81. On the question of testing, the fact that most of the settlement was recovered when water loading had been removed suggested that the ground was porous, and the conclusion might be drawn that the majority of the ground under this tank was sand or gravel of varying densities. For this reason Mr Bullen had been puzzled by the data given in Appendix I. For example, in borehole No. 10, medium brown grey mottled clay with sand lenses did not appear to be a clay but a sand, perhaps with some clay in it; this was supported by the fact that immediate undrained triaxial tests showed a zero cohesion and $\phi = 30°$. Surely this suggested a granular material, not a clay. Was a consolidation test appropriate on such a material?

82. Mr Bullen said that taking the data in Fig. 3 for pile No. 7, he had re-plotted the loading diagram (Fig. 14) and had concluded that this pile was supported largely by friction. On the assumption that the pile was supported at its toe entirely, the effect of the load on the head would be to cause deflexion of the head along the broken line. If, on the other hand, the pile were supported in friction, then the settlement at the head of the pile would follow another line which, as could be seen, but the plotted line at about 140 tons.

83. Making use of the remaining data in the diagrams, and applying a rule which he had recently suggested[2], the quake at the toe of the pile under final driving appeared to be about 0·1 in.; this gave a refusal drop of about 4¼ in., corresponding to a toe resistance of 71 tons, which added to the 140 tons friction gave a total resistance of 211 tons; this corresponded to the 210 tons actually applied to the pile. The interesting thing about these figures was that the friction of 140 tons was equivalent to about 0·87 ton/sq. ft, which was very high for clay but seemed to be possible in sands or gravels. The conclusion therefore seemed to be that this pile was driven in sand and gravel, and, in fact, carried two-thirds of its load in friction. A similar analysis for pile No. 37, which was only about one-half as long in the ground, although it was pitched 44 ft long, gave corresponding figures of 70 tons in friction and 60 tons toe value. Once again the rate of frictional resistance was 0·87 ton/sq. ft; and as could be seen from the driving records, this pile was driving more easily on completion than pile No. 7.

84. The broad conclusion, therefore, was that the greater support for the long test pile was in friction, and Mr Bullen wondered if the piles under the rotating arm base which were of reinforced concrete 50 ft long and 14 in. × 14 in., were also supported largely by friction. If their load was being carried in friction, would not the effect of the surrounding sheet-piling be to limit their load-bearing capacity to the frictional support of the box of steel piling?

85. Would the Authors explain how they had reached the conclusion that bearing piles would be of benefit generally? Further, if the base slab rose after loading, would the piles rise too, or would they be put into tension?

86. A further detail regarding the test piles was of interest. Calculation to find the stresses at the heads of the timber piles when being driven by a 4-ton hammer with a 4-ft drop suggested that the head stresses could have reached more than 5,500 lb/sq. in. This was very near to the ultimate strength of greenheart, and therefore Mr Bullen imagined that the pile heads would have been damaged during driving. Could the Authors say if that was so?

87. Finally, what had happened to the test piles ultimately? Had they been cut off or withdrawn? If withdrawn, would the Authors give details of the extraction forces, since they would be very useful in checking the frictional forces acting upon the piles?

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[2] F. R. Bullen, "Phenomena connected with the settlement of driven piles". Géotechnique, Lond., vol. 8 (1958), p. 121.

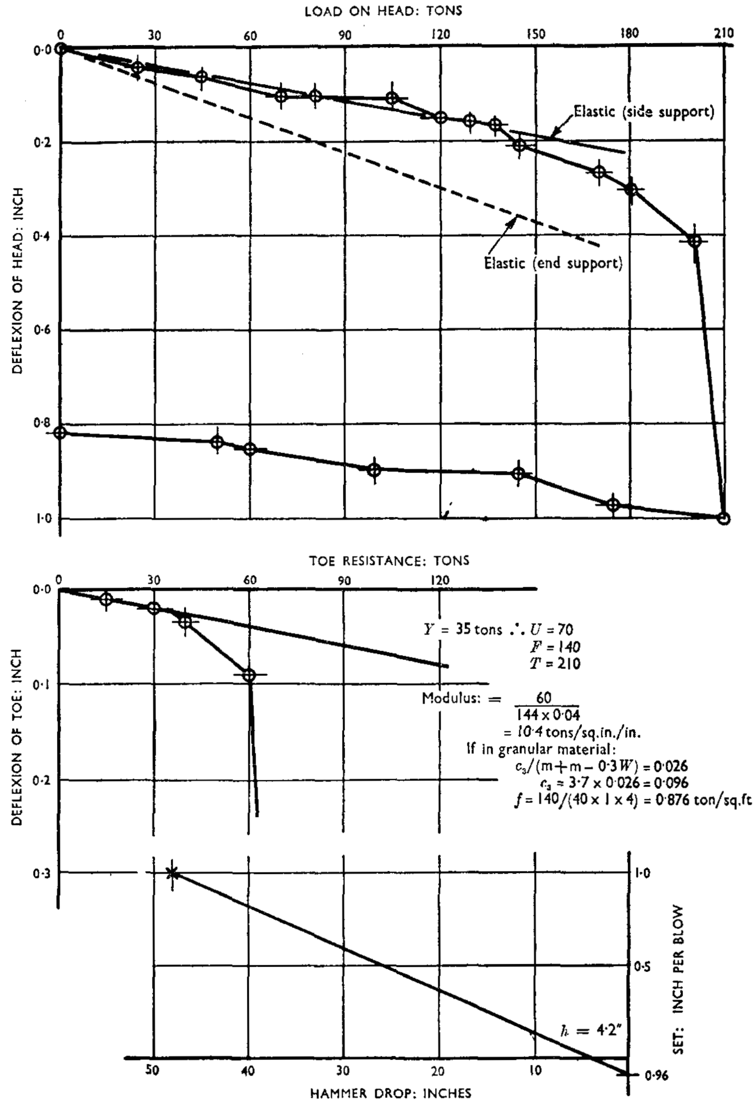

FIG. 14.—HASLAR TEST PILE No. 7 (12-IN. × 12-IN. GREENHEART, 47 FT LONG)

D. K. Wainwright (Site Agent, Messrs Trollope & Colls, Ltd) said that the principal components of the contract period had been as follows: Stage 1, preliminary work and piling, 8 weeks; construction of foundations, walls and floor slab, 52 weeks. Stage 2, the water test, 17 weeks. Stage 3, erection of steelwork, 20 weeks; completion of tank superstructure and auxiliary building works, 52 weeks. Within the period allotted to stage 1, the consecutive specified delays subsequent to the casting of the last bay of the wall foundation amounted to 30 weeks, made up of five periods of 6 weeks. After the last wall foundation had been cast there had been a 6 weeks' delay before the last gap between the last two foundations could be cast, and that had affected the last bay of the lower floor slab. Following that there had been two periods of 6 weeks on the lower floor, then the tanking, and then two periods of 6 weeks on the upper floor, making altogether 30 weeks delay after casting the last wall foundation.

89. It would have been possible to carry out the work in less time than the specified delay which governed the completion of each stage, so that the delays had actually regulated the programme. The result was that, including the periods for the water test and steelwork erection, the progress had been definitely limited for 15 out of the 34 months.

90. On the tank-wall shuttering the use of steel-channel walings had been necessary to span the distance between the buttresses. With the 18-ft lift employed, it was obvious that a maximum pressure had to be allowed for. The buttresses were 2 ft 3 in. wide and were splayed at the intersection with the wall. The walings on the opposite face spanned 6 ft 6 in. between tie bolts. The walings had been drilled before the shutter panels had been fabricated, to allow for the various positions of the studs required, for economic usage during the job. The sequence of movements of each panel had therefore to be predetermined.

91. With a clause in the specification requiring the contractor to cast the wall bay not later than 7 days after casting the foundation, care had had to be taken to avoid casting a foundation to which wall shutters could not be made available, and that had sometimes imposed restrictions on the progress of the job; but quite often it had been an incentive to spur on the wall shuttering. The principal difficulty in the erection of the large panels had been caused by their stiffness and stability which had been such an asset when the panels were finally in position and the bolted joints had been made firm.

92. In both design and erection the splayed junction between the battered face of the wall and the buttress had presented great problems. In an attempt to simplify the erection, an effort had been made to strike and fix buttress panels and adjacent wall panels as one unit; but, owing to minor errors in the alignment of the upstands on the foundation and slight distortion in shuttering forms when lifted, the panels could not be fitted into the internal angle. To minimize the alterations required to adapt the panels to suit the various positions, the bays had not been cast consecutively, and arrangements had been made to avoid concreting adjacent bays at the expansion joint with less than 4 weeks' delay. The expansion joint had obviously been well designed, but there was no point in adding to the movement which it had to accommodate by concreting the adjacent bays quickly.

93. Construction proceeded at the rate of two bays—wall and foundation—per week, determined by the usage of shuttering rather than the output of concrete. The average volumes were 60 cu. yd/day for the foundations and 40 cu. yd/day for the walls. All the scaffolding required for steel fixing, shutter erection, and concreting, had been in prefabricated units lifted into position by the derrick cranes.

94. The rate of placing the concrete had been greatly restricted by the small size of trunking which had had to be used. It had been purpose-made from 16-gauge steel sheet with a rectangular cross-section 8 in. × 5 in., that being the greatest size which could be put down. The walls were 11 in. wide at the top, but the cover and the reinforcement reduced the width to 5 in.

95. The floor slab area, 400 ft × 200 ft, was theoretically level, without any sumps, with the outlet pipes on the same level. The removal of rainwater had therefore been a problem. The best method was by suction with a 2-in. pump connected to the centre

of a 1-in.-thick steel plate, 9-in.-sq., lifted ¼ in. off the concrete. The impregnated cane fibre in the expansion joint had not been properly dried, because they had been unable to find a way of doing it. They had tried passing a small pipe down through the cane fibre and pumping, and that had worked, but it had been almost impossible to get it completely dry, and the adhesion of the bitumen jointing had been impaired.

96. In drilling the bolt-holes for the wavemaker guide rails the difficulty had been to ensure the accurate location and direction of the holes. The bolts had been fixed with caulked lead rings and were not adjustable in the hole. They had used a pneumatic rock drill mounted on a 4-in.-dia. tube fixed parallel to the wall. In a few instances where reinforcement was encountered, nothing had dealt with the combination of steel and concrete more effectively than tungsten carbide bits, although they had often been damaged in the process. They had tried a diamond core-cutting drill, but it suffered from difficulties in flushing out a hole drilled horizontally.

Mr E. H. Dickson (Managing Director, Caswell Cranes & Erection Ltd) said that his connexion with the contract had been concerned solely with the erection of the roof, excluding any work in connexion with welding. His firm had been faced with an unusual problem, since they had not been able to put any lifting equipment of the more desirable type on the area immediately below those parts of the roof structure which had had to be welded on the ground and subsequently erected. They had been unable to assemble each truss completely on the floor of the tank because the span of the truss was considerably greater than the distance between the side walls, and the angle to which the trusses would have had to be laid would have made necessary the installation of 15-ton cranes outside the tank to provide adequate cranage to transport from the welding jigs and erect. Such cranes would have had to travel on tracks with powered bogies. Because of constant movement under load the quality of the required temporary tracks, and the cost of installing such cranes, would have been out of all proportion to their service to the contract although it would have simplified erection procedure. They had therefore had to use 5-ton Scotch-derrick cranes, on 30-ft gabbards, so that the jibs could move unrestricted by the eaves of the building. This capacity was sufficient to cope with the largest piece of structure other than half-trusses and also sufficient to handle the derrick poles. Each girder had been laid out in two parts on the floor of the tank, each part being one panel less than a half-truss. The two centre panels had then been built on to one part, increasing its weight from 10 tons to 15 tons; hence the need for two derrick poles working from the floor of the tank. This method of erection had left the floor of the tank clear and thus provided the necessary area on which to build the half-trusses and weld them. Had his firm tried to put major items of tackle into the tank they would have been faced not only with the problem of getting the equipment in and its subsequent removal, but also with the problem of spreading the load on the floor. The finishing bays could not have been completed by such tackle because of the space it would have occupied.

98. The temporary support of the trusses had been by means of scaffold towers supported by runners. By simple guying they had been able to secure the half-trusses in reasonably accurate alignment, and no degree of movement had taken place during welding such that any really adverse effect had been produced. Because of the accuracy with which the short upstand columns had been installed, there had been little difficulty in aligning the bases of the trusses.

Dr T. P. O'Sullivan (Partner, T. P. O'Sullivan & Partners, Consulting Engineers), referred to the original proposals for the roof design. Although § 40 mentioned "riveted steel frameworks" for the conventional truss and northlight designs, it was not intended in these to exclude the use of welding or partial bolting.

100. Fig. 15 showed nine of the forms of conventional truss which had been considered. He understood that, subject to the economic conditions being satisfied, the northlight type (schemes 1F, 1G, 1H, and 1J) would have been most acceptable, but

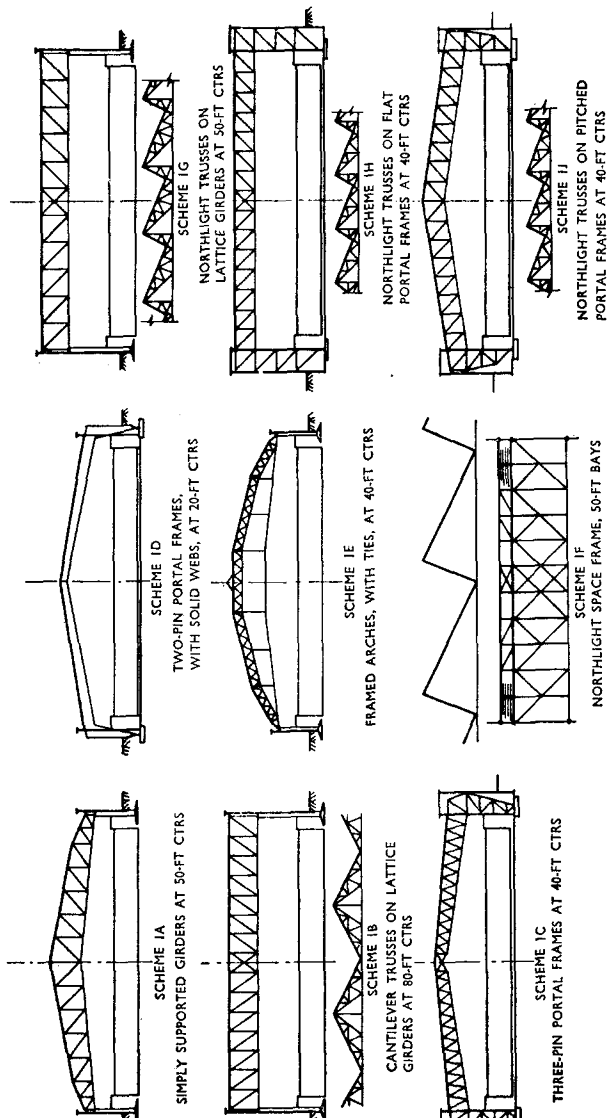

FIG. 15.—ALTERNATIVE SCHEMES IN STRUCTURAL STEEL

investigation had shown that the cost would have been about 20% more than for the non-northlight types. The weights per square foot were:—

Scheme	1A	1B	1C	1D	1E	1F	1G	1H	1J
Weight (lb/sq. ft)	12	13	13·5	15	12	14	12·5	12·5	12·5

The weight per square foot did not, of course, in itself, decide the economics of the issue, because the steelwork constructor would quote a different price per ton according to the work involved. The most economic types had been considered to be 1A and 1J, in which the weights of steelwork were approximately the same. It would be seen in Fig. 10a that the form of scheme 1A, although not its structural sections, had been adopted. Scheme 1J, although the most economical of the northlight types, had not been acceptable because of the additional incidental work required.

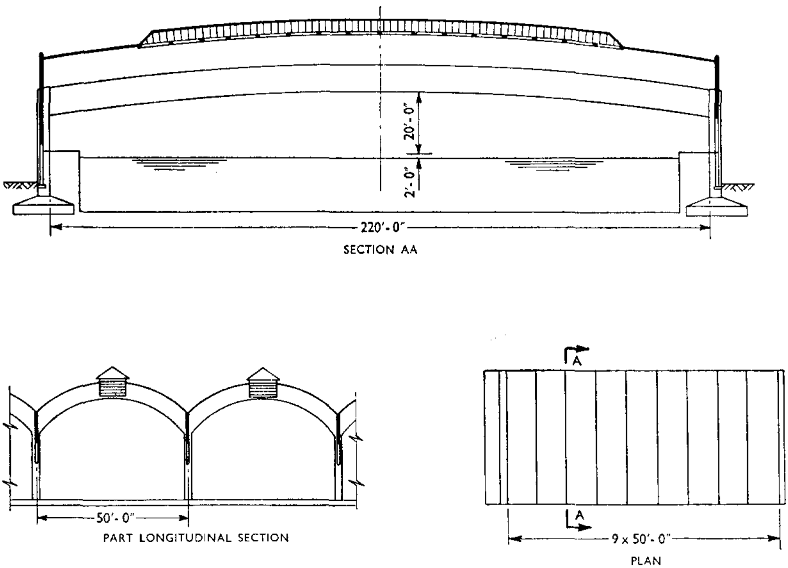

FIG. 16.—SCHEME FOR CONCRETE SHELL ROOF WITH PRESTRESSED EDGE BEAMS

101. Reference had been made to the possibility of a concrete-shell type of roof. The barrel-vault scheme shown in Fig. 16 had in fact been considered but would have cost about 50% more than the most economic scheme. Since the investigations had had to be done quickly, it was possible that the scheme shown in Fig. 16 was not the most economic possible concrete design, but even this somewhat cursory examination had shown that in this instance concrete had to be ruled out on economic grounds apart from anything else. Fig. 17 showed a scheme prepared by Professor J. F. Baker and Dr B. G. Neal. The Authors had mentioned that it would have shown a saving of 100 tons of steel and Dr O'Sullivan believed that that was dependent upon the adoption of a load factor of 1¾. He thought that however to make the comparison fair, it would be better to consider the scheme based on a load factor of 2; the saving would then be halved. Even so, if that scheme had been adopted *in toto* there would have been an overall saving in cost of 5%.

102. The plastic design had resulted in a very simple framework composed of 24-in. × 7½-in. joints, welded at the ends, with 6-in. × 3-in. I-section purlins providing the stiffening.

103. As in the other portal-frame designs the plastic scheme would, because of the

nature of the site and foundations, have required a continuous pre-stressed tie, which might, in the event of settlement, have caused some complications. No doubt they could have been overcome, and in any case, much careful thought had been given to the problem.

104. Dr O'Sullivan had been aware that the use of tubular members for trusses showed a slight saving, particularly on maintenance, over rolled steel sections, but could not understand how this saving had amounted to the £25,000 mentioned in § 40. He felt that great care should be exercised before forming any general conclusions about

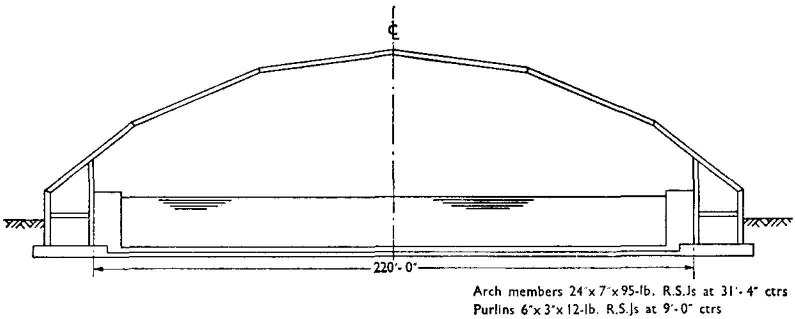

FIG. 17.—ARCH DESIGNED ON PLASTIC THEORY (BAKER AND NEAL)

the economics of the various schemes, because, apart from the question of weight (the tubular scheme was 70% of the weight of the conventional structural steel scheme), there was the question of the tonnage price the steelwork contractor or manufacturer would quote. He might decide merely to cover his costs or to add 25%; there might be questions of prestige involved.

105. While agreeing that in the Haslar project the tubular scheme had been the most economic, Dr O'Sullivan felt that great care should be exercised before assuming that a tubular scheme was necessarily the right solution for long spans.

Mr G. B. Godfrey (Tubewrights Ltd) observed that it was a long time since a Paper describing a large tubular structure was presented to the Institution. Steel tubes showed their greatest economy as structural media in tall structures, such as radio masts, where the allowable form factor for wind loads was far more favourable than that for conventional sections, or in large-span structures, where the light weight of the tubes considerably reduced the dead-load/live-load ratio, which was normally high in such structures. The main Haslar girders each weighed 23 tons, but a competitive girder in rolled sections weighed at least 40 tons. This must have contributed to the overall economy of £25,000 revealed by the Authors.

107. Mr Godfrey then described the design and fabrication of the main tubular steelwork for the Haslar roof. The main booms were butt-welded together, while the ends of the branch or web members were saddled and welded directly, without gusset plates, to the booms. The saddling operation could be carried out manually, by wrapping a brown-paper templet around the tube, marking off, and then cutting. He then showed details of a German machine, made by Müller of Opladen. The machine embodied three levers which were adjusted to suit the diameter of the branch, the diameter of the boom, and the angle between them. An operator controlled the speed of rotation of the tube, but the movement of the torch producing the profiled end was entirely automatic.

108. Sometimes arrangements had to be made to safeguard the stability of the walls of booms when branch members carried heavy loads. Such strengthening was required above and below the $8\frac{5}{8}$-in-dia. post at the first panel point shown in the top left-hand

corner of Fig. 10a. Short lengths of tube, 1 in. in wall thickness, were inserted in the top and bottom 16-in-dia. booms, which were otherwise $\frac{9}{16}$ in. or $\frac{7}{16}$ in. thick, respectively. The bottom joint is shown in Fig. 18. An alternative arrangement to ensure stability was to insert a diaphragm, analogous to a transverse stiffener in a plate girder, as shown in Fig. 19. This device required only one butt-weld in the boom.

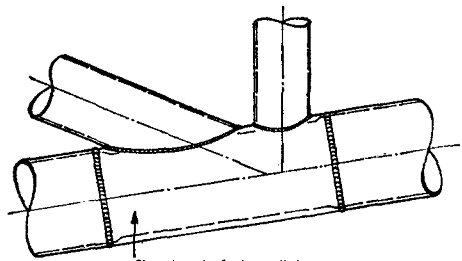

FIG. 18

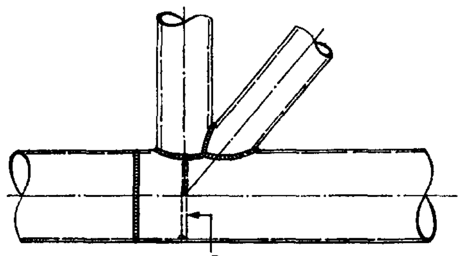

FIG. 19

Mr Otto Bondy (Welding Engineer, Ship Department, Admiralty, Bath) said that his department had been closely associated with the inspection and non-destructive testing of welding. In § 42 two makes of electrode, used in workshop fabrication, were mentioned, the first of the rutile type, the other of the low-hydrogen type. Their choice had been in accordance with good practice in the welding of ships' hulls, and the type of electrode specified depended on the thickness of the material, on the degree of restraint, on the quality of the parent metal, and on other considerations. Where the tubular joints were mainly in compression there had been no need to take special precautions which would involve additional cost, such as the use of low-hydrogen electrodes and pre-heating.

110. In § 46 it was stated that some of the 30,000 welded joints had involved eight radiographs each. For example, a 16-in-dia. tube might have a circumferential joint more than 4 ft long, and that would be split up into segments for radiographing, apart from repeat exposures after welding repairs.

111. Non-destructive testing was being applied to welded steel structures on a steadily increasing scale, and this fact should be taken into account in the design stage. That had not been so with riveted construction, where the soundness of the joint had been, rightly or wrongly, taken for granted. The Authors' reference to the haunch

boxes in § 44 was a case in point. Both the welding and the non-destructive testing of those boxes, of 1-in-thick plates, had given rise to difficulty. In § 44 the Authors had drawn attention to the possibility of resonance, which might be expected if the wave-making machinery excited vibrations of certain frequencies in the welded steel structure, and it was stated that tests made after the roofing was complete had shown that the natural vibration period was 0·2 sec. Had the expectation of resonance been confirmed in service, and, if so, what precautions had been taken?

Mr E. Lister (Research Manager, Stewarts & Lloyds, Corby) said that various methods were used for non-destructive examination of welds in tubular construction and showed a number of slides illustrating some of them.

113. Radiography was very widely used, X-rays and γ-rays from radioactive isotopes both being employed; for site work isotopes were usually preferred because of their portability. In one technique the isotope was placed on one side of the tube and the film on the other, the beam of γ-rays being directed through both. The beam was offset from the weld at the near side so that the shadow of this part of the weld did not fall across the radiograph of the weld on the far side.

114. In some cases it was possible to get inside the tube, and then it was possible to employ the single-wall technique, which was often the better technique. The isotope was carried on a crawler which was pushed into the tube and positioned on the centre-line of the weld. In that case film could be wrapped right round the weld and the whole circumference dealt with at one exposure. It was essential to position the isotope accurately, and for that purpose a Geiger counter was used; it was mounted in a heavy lead block containing a narrow hole, one end of which was positioned directly over the weld, the end window of the counter being placed at the other. Only radiation passing through the centre-line of the weld and through the small hole on to the counter would be detected. By moving the crawler slightly backwards and forwards it was possible to find the position giving the maximum signal from the counter and thus confirm the correct position.

115. It was not always possible to employ radiography, and in such circumstances the ultrasonic testing technique could often be used to advantage. A beam of ultrasonic waves was directed into the weld, and any interference with this beam due to the presence of a defect could be indicated by a signal on the cathode-ray screen.

116. Another useful non-destructive testing technique consisted of using two coils to magnetize the length of tube between them. The tube was then sprayed with a magnetic powder suspended in fluid, and any free magnetism existing at a crack was shown up by the deposition of black powder at the edges, giving a pattern of the defect.

The following contributions were received in writing.

Eric Newton (an Assistant Engineer, Port of London Authority), referring to the investigations on different types of roof structure, remarked that similar investigations into the problems of structural medium and structural shape by the Chief Engineer's Department of the Port of London Authority in recent years had also proved the economy of welded tubular construction for medium- and large-span storage-shed roofs and had indicated that there was a further economic advantage in reducing to the minimum the number of stages in which the loads were carried to the foundations. Evidence in support of this statement was provided by the 125-ft single-span storage shed shown in Fig. 20. This shed had originally been designed as an umbrella roof structure in rolled-steel riveted construction, with 125-ft main girders spaced at 45-ft centres carrying the cantilever half-trusses. At the tender stage contractors had been allowed to submit additional tenders based on any alternative form of steel construction selected by them. The winning design had been for a tubular tied-rafter roof with trusses at 15-ft centres and single-tube purlins. The building was 300 ft long and had been designed to B.S.499, but the imposed load had been increased to 15 lb/sq. ft. The total weight of the roof structure above column-cap level was 2·8 lb/sq. ft of floor area and this design resulted in a saving of approximately £2,000 compared with another tender design for an umbrella roof structure in welded tubular steel, and approximately £4,000 compared with the original design of the umbrella roof in rolled steel sections.

118. Later projects, at present in the design or fabrication stage, had confirmed the economic advantages of the tied lattice arch and the bowstring truss. In the case of one shed, with a clear span of 200 ft and a length of 700 ft, the weight of the roof structure above eaves level, including purlins, longitudinal ties, wind portals, and intermediate wind bracing, was 5·3 lb/sq. ft of floor area. Welded tubular steel construction was used in the main, but rolled sections were preferred for certain members (§ 120).

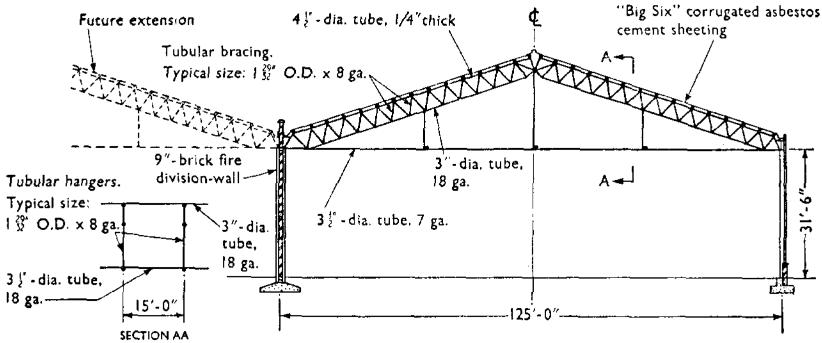

FIG. 20

119. It seemed doubtful if the arrangement of main trusses, secondary trusses, and tertiary trusses adopted at Haslar had led to the most economic solution. A spacing of 22 ft 6 in. for the main trusses, with trussed or braced purlins spanning between the trusses, might have led to a considerable saving in steel and cost, even taking into account the provision of supports for the intermediate trusses, and would also have reduced the weights of the trusses and the consequent erection problems. No doubt the designers had been influenced by considerations of overall rigidity, but it was difficult to see why a simpler arrangement—such as the one outlined above—combined with an efficient wind portal system at the gables and intermediate wind bracing, would not have sufficed.

120. It appeared, moreover, that the compression boom was stabilized at unequal centres laterally and vertically and the question then arose whether or not a round tube was the most suitable section as a strut under these conditions. In the case of the P.L.A. sheds mentioned earlier it had been found that under similar conditions the use of a steel channel laid flat had considerable advantages over a tube, and a channel was now used as a section for ties, subjected to load reversal under certain wind conditions, as well as for struts, where it was not practical to stabilize the member evenly in two directions. Where a round tube was used as the main compression boom or rafter, care was taken in the design to stabilize it at equal centres both vertically and laterally. Together with the use of rolled steel sections as cantilever columns this had led to the development of a composite system of welded construction embodying both round steel tubes and rolled steel sections where each could be used to its best advantage.

121. A problem encountered with simply supported large-span tubular lattice trusses, having booms of uniform diameter and thickness, was how to avoid waste of metal over the greater part of the span. The Authors had deepened the main trusses towards the centre, and had thus achieved some equalization of stress. A tied arch or bowstring, and particularly one having a parabolic or approximately parabolic rib,

would seem to give the most uniform distribution of load in the two principal members. It was possible to vary the diameters of the tubes in long booms, and special taper tubes were now readily available as junction pieces. Surprisingly, however, it had not been found practical to make butt welds between tubes of equal outside diameters but varying thicknesses, which might entail the use of backing rings machined on one side. This seemed to be one problem that still awaited a practical solution by the tube fabricators, for taper pieces tended to be unsightly and were not always acceptable.

122. It would be interesting to know the weight and cost per square foot of floor area of the Haslar tank roof structure, and perhaps the Authors would also indicate what loading had been taken for the roof. Had any allowance been made in the design for lifting loads at panel points, and what provision had been made for dealing with wind forces acting on the gables? Did the Authors consider the use of Grade 20 steel for the main booms? It was sometimes claimed that with larger sections in the hot-finished seamless range, i.e. between $4\frac{1}{2}$ and 16 in. outside diameter the use of Grade 20 steel became increasingly economical, in spite of higher welding costs owing to pre-heating at the joints.

123. There was little doubt that welded tubular steel, when used with discretion, could make an important contribution to the economy of industrial roof design of all spans, and so long as the relative costs of different forms of steel construction did not change radically it was likely to become increasingly popular.

Mr N. A. Wootton (C. H. Dobbie and Partners, Consulting Engineers) noted that the recommendations made in the Code of Practice for the Design and Construction of Reinforced-Concrete Structures for the Storage of Liquids, limited the working stress in the reinforcing steel to 12,000 lb/sq. in., and asked why the decision to use Tentor bars had been made, since the economy usually obtained by working at higher stresses was lost in those circumstances. Had it been for the purpose of obtaining improved bond and therefore better crack distribution? In that case what would the hair cracks in the walls have been like had round bars been used?

125. Mr Wootton thought it was wrong to ascribe the development of hair cracks in the walls (§ 25) to the differential movement. Surely they occurred as a result of the restraint which prevented the relative movement due to the different rates of shrinkage of the walls and foundations. That problem was always present when the monolithic construction of the walls and foundations was necessary.

126. The 7-day maximum period between pouring the foundation concrete and casting the walls might have been sufficient to have eliminated visible cracks resulting from the contraction due to drying shrinkage only. In this case, however, where high temperatures had been recorded in the concrete, he estimated that at least half of the shrinkage (had the walls been free to move) would have been due to thermal contraction as a result of the concrete cooling to normal air temperature. On the basis of the shrinkage strain and the coefficient of thermal expansion given in the previously mentioned Code of Practice, and allowing for a temperature decrease of 45°F, the following figures were obtained for a 21-ft length of wall: drying shrinkage = 0·05 in.; thermal contraction = 0·06 in.; total contraction 0·11 in. (nearly $\frac{1}{8}$ in. which is quite appreciable). The use of smaller lifts for the walls might conceivably have reduced the temperatures attained in the concrete; steel shuttering would have allowed better heat dissipation.

127. With regard to the leakage at the bolt holes, Mr Little had suggested that this might have been due to a very small settlement of the concrete in the shutter after consolidation but prior to setting. This hypothesis, if proved, would require engineers to revise their ideas on the bond strength of all horizontal reinforcement bars in walls, and the thought that horizontal bars on water faces might be partially embedded in a water film was extremely disturbing.

128. Mr Wootton suggested that the tendency for a gap to occur between an embedded tie and the surrounding concrete was attributable to the vibration arising from the shutter as a whole, or to local vibration transmitted through the shutter after consolidation of the concrete around a bolt or tie. There was always a tendency for

shutter bolts to loosen because of nut rotation, and even with lock-nuts there was normally sufficient clearance on holes for the bolts through the shutter to allow "chattering" to occur; this movement, however slight, could produce an annular space around the ties sufficient for the passage of water. Puddle flanges on the ties might not entirely have cut off the water paths for the same reason.

129. The use of bolts with proper sleeves, and the subsequent filling of the holes with one of the expanding cements might have given a more satisfactory job. The bolts would not have been so expendable as the embedded ties, and this saving would have helped to offset the cost of filling the holes.

130. The leakage caused by the hair cracks, and at the bolt holes appeared to have been of a minor nature and easily remedied. Ways of reducing cracking and obviating leakage at bolt positions had been suggested. The fact remained, however, that with vibrated concrete there was an increasing tendency to use large shutters and these had economic advantages, which no doubt more than offset the cost of any remedial work which might arise from their use, or contractors would not be inclined to employ this technique. Would the Authors, if faced with the same project again, carry out the wall construction in the same manner?

Mr R. J. Halstead (Resident Engineer, Kubani Bridge construction, Zaria, N. Nigeria) noted from § 22 that, because of the high temperatures recorded in the tank-wall concrete during curing, the shuttering had had to be stripped ahead of schedule during the warmer months of 1955. Was 115°F the highest temperature reached? Was the shuttering stripped early in order to accelerate cooling or to prevent the temperature rising above 115°F? That was to say, did the Authors consider that the extent of the shrinkage in the concrete depended on the maximum temperature attained by it during cooling or on the rate of cooling or on both?

132. The temperature of the 1:2:4 concrete in a pile cap cast recently at Zaria in Northern Nigeria had risen to 115°F after 14 hours and to 122°F after 26 hours. The maximum recorded air temperature in the same period was 101°F. Although prevention of shrinkage was particularly important in a water-retaining structure, it might, nevertheless, be good general practice in the tropics to strip vertical shuttering before the normally specified 3 days. Did the Authors agree that what might have been an extraordinary practice in the United Kingdom during the hot summer of 1955 might be advised as a regular practice in Nigeria?

Mr Robert Weir (Technical Director, Trollope & Colls Ltd) wrote pointing out the importance of observing the recommendations contained in the Code of Practice[3] for the design and construction of such structures.

134. He threw some doubt, however, on the value of contraction gaps, and suggested that if a suitable limitation was to be placed on the length of concrete to be placed in one bay and if the concreting was carried out in alternate bays, then the contraction gaps as such were unnecessary.

135. He emphasized the need for adequate supervision during construction so that the design should be interpreted correctly and that only high-class materials and workmanship were used.

The Authors, in reply, said that the late Dr R. W. L. Gawn, D.Sc., C.B.E., M.I.N.A., R.C.N.C., former Superintendent of the Admiralty Experiment Works, Haslar, had been responsible for the conception of the scheme. Dr Gawn had not lived to see it complete, but he had had the satisfaction of carrying out the first turning experiments in the tank in the autumn of 1956, while the tank was undergoing water test.

137. In reply to the President's question, the early scheme with plastic-designed

[3] "Code of Practice for the design and construction of reinforced-concrete structures for the storage of liquids". Instn civ. Engrs, 1950.

portals had not been adopted because it was too far in advance of current practice. With such a design, large deflexions would have occurred in the roof members. The uneven settlement of the foundations would have increased the distortion of the roof; while the roof framing could have been designed to cater for such distortion, it might well have caused difficulties later in both the roof covering and the brickwork. Dr O'Sullivan had suggested a load factor of 2, but the Authors considered that in plastic designs of that kind a load factor of 1·75 was appropriate; the general argument for using such a load factor had been set out by Professor Baker[4]. The Authors also considered that in a structure like the manœuvring tank the design should be such that under the worst conditions the stress in the steel nowhere reached the elastic limit. With large settlements in the foundations it would have been difficult to satisfy this condition.

138. The Authors agreed with Mr Bullen that some of the upper materials of the site were alluvium and plateau gravel, and not part of the Bracklesham beds; as mentioned in § 4, part of the site was in old made ground lying over mud. The extent of the ground heave when the greenheart piles were driven varied.

139. The piles had been driven to a calculated resistance of 100 tons; the loading test on pile No. 37 had confirmed this. The calculations were further supported by the loading test on pile No. 7, which had been driven to a calculated resistance of 200 tons. The piles were not intended to act like reinforcement in concrete; they had been put in to deal with the poorer types of foundation as shown in Fig. 3c, test slabs A and D. On the latter a loading of 2 tons/sq. ft produced a settlement of 2 in. In the poorer type of ground the pile had been stiffer than the ground and the foundation loading had pushed the pile further into the ground. Because the pile could carry only 100 tons the loading redistribution shown in Fig. 4 had occurred. The Authors agreed that the piles were mainly supported by side friction; Fig. 2 showed they were largely driven into sandy material. Unfortunately, when the base slab had risen after the tank had been emptied, there had been no way of telling whether the greenheart piles were put into tension; but whatever had happened the foundation design had proved satisfactory. In comparing the actual settlements observed in the tank with the measurements made in the slab and pile tests, allowance had to be made for consolidation of the ground below the level of the pile shoes as well as in the first 20 ft, the slab tests having only measured shallow effects immediately under the level of the foundation concrete.

140. The Authors thanked Mr Bullen for applying his new method of pile-settlement analysis to test pile No. 7 and for separating: (*a*) the elastic compression of the pile arising from skin friction; (*b*) the elastic compression of the ground under the toe of the pile; and (*c*) the plastic flow in the ground under the toe of the pile. That method of analysis should prove helpful in future. It was interesting to compare the temporary compression in the pile and the quake in the ground (c_2+c_3) measured during driving (0·15 in.) with that calculated by Mr Bullen for the quake only (0·096 in.).

141. No appreciable damage had been caused to the heads of the greenheart piles by normal driving. The Authors would not have expected it, for the ultimate strength of greenheart when wet was about 10,000 lb/sq. in. rising to about 15,000 lb/sq. in. when air-dried[5]. However, after test pile No. 7 had been driven to the required set of 1 in/blow it had been allowed to stand for 24 hours and had then been re-driven, the new set being 0·23 in/blow, and the temporary compression (c_2+c_3) rising from 0·15 in. before standing to 0·32 in. afterwards. With such driving the head of the pile had been crushed; but the stress had been double that arising in normal driving. Since the test piles had been cut off well below foundation level, no information was available about their resistance to extraction.

142. It was interesting to note that averaging the figures for unconfined compressive

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[4] J. F. Baker, "The design of steel frames". J. Instn struct. Engrs, vol. 27 (1949), p. 397. See p. 422.

[5] "The strength properties of timber", Appendix A. D.S.I.R. Forest Products res. Bull. No. 28 (1953), H.M.S.O., Lond.

strengths given in Appendix I for the materials penetrated by the 20-ft greenheart piles gave a figure of 20 lb/sq. in., i.e. a shear strength of 10 lb/sq. in. The area of the sides of each of these piles was 20 ft $\times$ 4 ft $=$ 80 sq. ft, which at 10 lb/sq. in. $=$ 51 tons. This was of the same order as the 70 tons quoted by Mr Bullen in § 83, although he had calculated that figure by a totally different analysis. The higher figure was more reasonable considering the friction arising from the sands.

143. In § 81 Mr Bullen had commented on the apparent conflict of the test results given in Appendix I for borehole No. 10 at +2·5 O.D. The records had been checked and the figure of $C=0$ lb/sq. in. and $\phi=30°$ with a consolidation coefficient of 0·0025 sq. in/min had been confirmed; it had been concluded that the part of the sample used for the triaxial test had been more granular than that used for the consolidation test. That could readily have occurred in very variable strata. The Authors considered that the consolidation of the cohesive materials under the tank was the cause of the small but continuing settlement which had occurred throughout the water-loading test of the tank.

144. The area of the outside surface of the ring of sheet piles under the rotating arm base was considerably greater than the surface of the four reinforced concrete piles; the sheet piles therefore actually increased the load-bearing capacity of the foundation.

145. Fig. 21 showed the settlement that had occurred in the south-east wall of the tank near station 2 (see Fig. 9a) after the tank had been finally filled for use, the October 1958 settlement being $1\frac{3}{16}$ in. compared with $\frac{1}{2}$ in. during the water test. On the other walls of the tank settlements were of the same order as those during the water test. No satisfactory explanation had been found for the increased settlements on the south-east wall.

146. In view of the part Dr O'Sullivan had had in considering various possible schemes for roofing the tank, the Authors were pleased to have his contribution to the discussion with information on the schemes shown in Fig. 15.

147. Mr Bondy had questioned the need for pre-heating and low-hydrogen electrodes where the welds were in compression members. Where a high standard of welding was required such precautions were wise, especially for joints kept in position for welding by guys and other temporary supports. Only by insisting on the better practices could high quality work be maintained; and to relax the requirements in the control of site welding was comparable to relaxing the requirements in the control of the formation of construction joints in concrete.

148. Because the wave-makers had not yet been in use in the tank it had not been possible to observe whether the roof steelwork vibrated in resonance with them.

149. Mr Lister had said that sometimes the isotope for radiography could be carried into tubes on a crawler. That had been done sometime at Haslar.

150. Mr Eric Newton had raised the question of the economics of the roof design. Dr O'Sullivan had shown one design (Fig. 15, scheme I.E.) that had tied rafters at 40-ft centres. The increased cost per ton for steelwork of that type made the design uneconomic; but it would have been instructive to have had a comparable design and estimated with the trusses at 15-ft centres, as in the Port of London. The steelwork in the tubular design used at Haslar weighed 36 tons for each 45-ft bay, or 8·2 lb/sq. ft compared with 5·3 lb/sq. ft for the slightly smaller span in the P.L.A. building. The roof structures, however, served different purposes; that at Haslar carried an insulated roof and working platforms. But by using tubes of a higher grade steel, and as a result of experience gained since the Haslar roof steelwork was designed, the weight of steelwork could probably be reduced. Because the use of Grade 15 tubes was the result of competitive design and tendering, the use of Grade 20 tubes had not been investigated by the Admiralty.

151. The point that tubes in compression ideally were best stabilized at equal centres laterally and vertically was important in considering the best section for members. Wind on the gables was catered for by horizontal wind trusses; one could just be seen in Fig. 12.

152. The Authors agreed with Mr Wootton that in tank construction the advantages gained by higher stresses permissible by the use of Tentor bars were largely reduced by

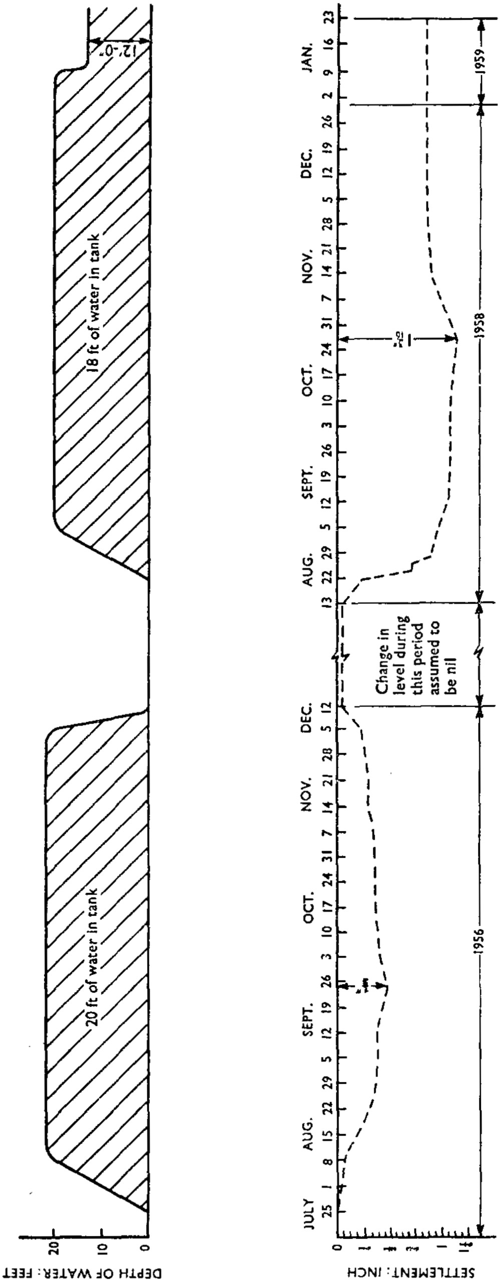

FIG. 21.—SETTLEMENT IN SOUTH-EAST WALL AT STATION 2

other considerations. At the time Tentor bars had been more readily available than other reinforcement, and that had clinched the decision to use them. The restraint Mr Wootton had referred to in § 125 had caused differential movement between the concrete in the walls and the concrete in the bases of the centres of the wall bays; if there had been no movement there would have been no crack.

153. The fact that water found its way past some of the embedded ties and appeared at the bolt holes had surprised the Authors. That was possibly due to a tendency for grease to remain on the embedded tie; but more likely, with such a high lift it resulted from a tendency for settlement of the concrete, however well vibrated, leading to a minute cavity under the tie. A similar phenomenon had frequently been noticed in flat slab construction in which top steel was present, where a regular pattern of cracks had developed over the top steel, owing to minor settlement of the concrete. The embedded ties themselves were torpedo-shaped with threaded holes at each end for the bolts to fit into, and had no puddle flanges. Through bolts were not permitted by the specification.

154. Dampness at some of the construction joints arose possibly from a tendency for hair cracks, caused by contraction near the joint, to form a path round the embedded water stop. Banks had commented on this possibility in his Paper on the Allt-na-Lairige Dam[6].

155. The Authors shared Mr Little's view that placing the walls in 3-ft lifts might have reduced cracking. Another possible method was to induce cracking deliberately in predetermined places by forming a central duct in the wall, and then to grout this after cracks had formed. Assuming, however, that concreting the wall in one lift was desirable, which obviously carried with it the necessity of having a concrete mix sufficiently workable to be placed under those conditions, it seemed that the only way of reducing this tendency to crack was to have shorter bays and to cast the wall as soon as possible after its foundation. In practice, with the varying shapes and sizes of the bays, the contractor found that even the 7-day period specified was tight at times.

156. Mr Halstead had asked about temperatures in the concrete during curing; 115°F was the highest temperature recorded. The Authors were generally in favour of stripping shuttering as early as possible to enable curing measures to be put in hand without delay. A possible exception was in cold weather.

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[6] J. A. Banks, "Allt-na-Lairige prestressed concrete dam". Proc. Instn civ. Engrs, vol. 6, p. 409 (Mar. 1957). See p. 423.