Paper No. 6305

Shoreham Harbour development †

by

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Discussion

The Author, when introducing the Paper, called attention to some errors. In § 10, the length of the vessels which could pass the lock at the entrance to the canal should be 228 ft, not 200 ft. In § 31, the diameter of the concreting pipes should be 6 in., not 4 in. In § 39 the diameter of the rods should be 1½ in., not 1¼ in. The correct figure was given in Fig. 4.

Mr G. A. Wilson (Chief Engineer, Port of London Authority) said that in asking a number of questions he would, in some cases, quote equivalent practice in the Port of London, so that the record would be extended for later readers.

82. Since models dealing with problems of littoral drift were notoriously difficult he hoped the Author would be able to provide more information.

83. It would appear from §§ 15 and 16 that the site investigation had yielded information necessary for design; yet, later on, it had been necessary to re-design the walls. How did this come about? (Surveys were frequently made in the Port of London, and occasionally the wrong answer to a problem was arrived at; it was always instructive to examine the reasons.) If the clay were more widely spread over the site it might account for the movement which had taken place in the breakwaters.

84. In § 52 it was mentioned that the work had been stopped while new designs were prepared for the lock walls. What were the contract conditions? Mr Wilson thought that the "General Conditions of Contract" document, published jointly by the Institution, the Association of Consulting Engineers, and the Federation of Civil Engineering Contractors gave contractors a very fair deal, but, in view of the quantity of plant on the site while the job had been stopped, the contractor's claim for delay must have been interesting. He had often suggested that the Institution should collect cases of contract settlements, so that when the "General Conditions of Contract" was reviewed, as must happen in time, there would be some record on which to work. The Shoreham case could well be the first on that record.

85. There was mention in § 45 of small-scale model experiments made to find out how the discharge culverts should be designed and the length of the apron required to prevent scour. Fisher had described¹ a similar model which had been made in the Port of London, from which the conclusion had been reached that the orifices from the culvert should be so arranged as to give a uniform flow across the end of the apron. If the culvert were so designed that there was a concentration of flow off the middle of the apron, scour would result. What was the Author's view of that, and had he taken any soundings to find out what was happening at Shoreham?

86. Massive steel and ballast structures had been used for the lead-in jetties; which one would hardly think would be kind to ships. In the Port of London it was customary to use timber, because it caused less damage to ships and was fairly easily repaired.

† Proc. Instn civ. Engrs, vol. 11, p. 285 (Nov. 1958).

¹ J. A. Fisher, "Reconstruction of the Gallions Lower Entrance Lock at the Royal Docks of the Port of London Authority". Proc. Instn civ. Engrs, Pt II, vol. 5, p. 136 (June 1956).

The Authority had recently adopted a new and calculated design in which the structure consisted basically of 12-in. $\times$ 12-in. greenheart piles, tied together in pairs to act as vertical cantilevers. The tops were braced together to form a strong but light structure which could absorb a considerable impact and was easy to repair if broken. It had been interesting to hear that the Author had thought of using timber with a concrete deck on it but in his opinion that would not be a suitable combination of materials.

87. Gates were a matter of daily concern to the engineers of the Port of London, and when they saw anyone doing something new or clinging to some old idea they were always very interested to know the reason. For operating gates a recent tender had shown that the cheapest method was electrical-mechanical. Why had the Author used oil-hydraulic? The specification for the oil-hydraulic ram and cylinder was surprising; it was a cast-iron cylinder with a gun-metal liner and a piston rod in high-tensile bronze. Mr Wilson considered that a cheaper and equally effective arrangement for the cylinder would be extruded high-pressure steel tube, which could be bought "off the shelf" with a 5% tolerance on the bore. With this a stainless-steel piston rod and mild-steel piston head could be used.

88. What sort of force was applied to the lock gates at Shoreham? In the Port of London it had been found that in general the force was far too large. That was attributed to the fact that the design of the cylinder depended on the diameter of the piston rod, which in turn depended on the slenderness ratio of the rod. It was as well to keep the force low, because with excessive force the gates could be easily strained.

89. There did not seem to be any provision for the time when the gates had to be overhauled and colliers had still to get into the berths. When an overhaul was necessary it would take two or three months. In the Port of London, where the maximum duty on a lock gate was about 7,000 movements a year, and sometimes as many as 30 in 24 hours, it had been found that greenheart was hardly sufficiently durable to give reasonable maintenance costs and consideration was being given to using metal-to-metal contact and avoiding the wear by introducing eccentric pivots.

90. Would the Author say a little more about the wear on the sheet piles at the Hamblin Groyne, what the piles were, how long they had been there, and what sort of conditions they had to encounter?

Mr P. A. Scott (Partner, Sir William Halcrow and Partners, Consulting Engineers) referred to the proceedings when the Shoreham Harbour Bill was put before Parliament. There had been a great deal of opposition. The proposals for very large alterations to an existing harbour were bound to upset and affect the interests of a great number of people. Mr Scott had been the engineering witness for the promoters; it had taken 20 foolscap pages of evidence from him to convince four members of the House of Commons that the Bill should go through, and half as much in the later hearing in the House of Lords.

92. The opposition had not been directed entirely to throwing the Bill out, but rather to ensuring that the people who might be affected by the building of the harbour would be protected. At least twelve authorities had registered fears that the breakwaters in particular would affect them. They had been represented by twelve counsel and the promoters by three. The biggest fears arose from the possible effects on the littoral drift. This had been of concern to the corporations of Brighton and Hove and to all the urban district councils east of the Shoreham Harbour entrance. The fear had been, of course, that the building of big breakwaters out into fairly deep water would mean that the movement of the shingle would be arrested and the beaches to the east starved. It was obvious that in spite of the assurance with which Mr Duvier (for the opposition) and Mr Scott (for the promoters) had given their evidence, in opposite directions, about what would happen to the shingle, they had a great deal still to learn. To Mr Scott, it was obvious that the theory that the groynes or breakwaters completely arrested the movement of shingle was not correct. Shingle must move in deep water, or there would not be a continuance of the shingle beaches to the east of the breakwaters and groynes which already existed on the South Coast.

93. The other subject which had caused great controversy was the sewage problem. In the vicinity of Shoreham harbour there were two sewers. The Shoreham authority discharged their sewage at the middle pier, and the Portslade and Southwick Sewerage Board discharged theirs through an outfall which, though not shown in Fig. 1, Plate 1, went through the word "Prince" in "Prince George Lock" in that Figure. These facts presented considerable problems, because the authorities said that the building of the breakwaters would disturb the carrying away of the sewage to sea. In the case of the one inside the harbour, it had been argued that the effect of the new breakwaters would be to reduce the velocity of the River Adur on the ebb tide, while in the case of the sea outfall it had been argued that the building of the breakwaters out to sea would deflect the tide which carried the sewage away into the Channel and that the sewage would be trapped within the arms of the breakwaters.

94. This led the authorities to investigate what in fact the then existing conditions at those sewers had been. The method of operation was that the sewage was collected in tanks during the periods of high water and discharged, by a man opening a valve, at a certain stage of the tide, when the ebb had started to run and the current was going from east to west down the Channel. The theory was that in that way the sewage would be carried away and dispersed in the Channel. Broadly, that was the system used all round the coasts of Great Britain, but it was a system which, a hundred years hence, would, he was sure, be regarded with horror as a barbarous practice. Unfortunately, local authorities, for some reason, were not prepared at the moment to face the cost of treating the sewage. A simple mathematical calculation would show what it would cost fully to treat all the sewage which was today discharged to sea in a raw state; but why the lucky inhabitants of seaside resorts, who made much of their money by entertaining people from inland, should not have to treat their sewage as those inland had to do was a mystery which had not yet been solved.

95. However that might be, this sewage question had been treated very seriously. A bacteriologist had been consulted and he had stated that the conditions on the beach were far from satisfactory, in that the sewage was not carried away quite as efficiently as had been thought. The Authority had naturally not been willing to extend the sewers if the conditions there had already been bad. However, being a public authority, they had taken a broad view and a new sewer outfall was to be built. What would happen to the sewage discharged at the middle pier had yet to be seen.

96. Among the other people who had been affected by the works or who thought that they would be, were the owners of all the wharfs inside the harbour, who feared that as a result of opening up the breakwaters there would be a greater wave action against their berths. Also, the authorities responsible for the Brighton-Worthing road had feared that it might be affected at two points, the one mentioned by the Author and one at the top of the bank to the north of the dredged harbour, where a slip had occurred. It had not yet been decided whether or not that slip had been due to the dredging, but the point had not been lost sight of in the planning stages. The subsequent investigation into the causes of the slip had not yet proved what had been the real reason for the movement.

97. The construction of the lock had required engineering skill of a high order on the part of the contractor in designing the constructional method, and this had been forthcoming in a high degree from Mr Prior and his associates.

Mr Jack Duvivier (Partner, Messrs Lewis and Duvivier, Consulting Engineers), who had been connected with the project in 1949, when Parliamentary powers were being sought and the onerous protective clauses referred to in § 6 of the Paper were being negotiated with the county and municipal councils responsible for the protection of the coast-line to the east, said that it had been the intention of the promoters that the western pier—which was to be the longer of the two—should be aligned in a south-easterly direction and extend approximately 500 ft seaward of the existing western pier head.

99. Before the works had begun a great quantity of shingle had accumulated against

the western side of the harbour entrance pier. The depth of water at the then existing pier head had been approximately 2 ft at M.L.W.S. and the depth of water at the entrance channel to the harbour varied between 4 and 6 ft below M.L.W.S. The proposed new western arm would terminate at a depth of approximately 10 ft below M.L.W.S., and it had been proposed to establish and maintain the new entrance channel to a depth of 9 ft below M.L.W.S. by dredging. The western pier would extend out into a depth of water beyond that in which ordinary beach movement took place and would therefore, he submitted, constitute an impermeable barrier in the path of the littoral drift. The shingle would gather in a bank against the western side of the new pier and, as it did so, the beach would advance seawards until a new beach had formed down to the toe of the pier. During this period no drift of shingle would take place past the harbour. It had been said in evidence before Parliament, and just repeated by Mr Scott, that shingle did in fact travel along the sea-bed in an appreciable depth of water, but that submission, in his opinion, was not supported by any evidence. The evidence and his own experience of these matters pointed to the contrary.

100. The Author had made a statement in presenting the Paper which was of very great interest, namely, that he had placed open boxes on the sea-bed beyond the ends of the breakwaters; when they had been examined at a later date it had been found that they contained no shingle, which seemed to prove fairly conclusively that there was no measurable depth of shingle along the sea-bed at Shoreham beyond the range of littoral movement, which extended only to a few feet in depth beyond M.L.W.S.

101. It had not been possible to offer any useful estimate on the period which would elapse before the condition of shingle spilling round the western pier was reached. Since the rate of flow of the shingle from the west was governed by intercepting groynes and the state of their repair, the direction and strength of winds, and other matters of complexity difficult or impossible to forecast, it had been obvious that no such estimate could be made. When the beach had reached the toe of the new west pier it would spill into the navigation channel, unless an anticipatory removal of the accumulation had taken place.

102. Further consideration had been given to the effect of this stoppage of the littoral drift on the beaches east of the harbour entrance. Those nearest to the entrance would first feel the effect of this stoppage by suffering a scouring of their shingle without getting any replenishment. These beaches would be successively denuded, and the last to be affected would be those fronting Brighton. It had been considered reasonable for the Brighton Corporation to ask for a series of safeguarding provisions under a special clause, and it had been considered at that time that this clause should require that all sand and shingle excavated or dredged within the harbour and not required in the construction or maintenance of the harbour works should be placed on or near to the east beach, where it would come into circulation; and that all sand and shingle removed from the west beach should be dealt with in the same way. Up to that time it had been the custom to excavate, from the west beach, considerable quantities of shingle, for sale as ballast, and it had been considered by the various authorities seeking safeguards that the west beach should be acquired by the promoters to enable them to control the disposal of the shingle in the years to come.

103. There had been considerable discussion at the time on how the authorities concerned with the protection of the beaches to the east were to know whether or not the construction of the new harbour works had had any effect on their beaches, and it had been proposed that a referee should be appointed and that he should be required to survey the beach slope and the sea-bed close inshore, both on the west side of the harbour and along the Brighton Corporation's foreshore, and, from such surveys, to calculate the changes in the contour or profile of such beaches and thus the quantities of material which had accumulated or been lost by scour, as the case might be. It had been felt that the promoters should themselves institute the first survey of the west beach immediately before the construction of the west pier took place, with the co-operation of the Brighton Corporation.

104. Mr Duvivier had no information as to the form in which the safeguarding

clauses had been finally agreed with the local authorities, and would be interested to know. Perhaps the Author could give a précis of the clauses.

105. Mr Duvivier then referred to the life of steel sheet-piling when subjected to the abrasive and percussive effects of wave-driven beach shingle. The rate at which steel sheet-piling was worn away by the scrub of shingle depended on many factors. The principal factors were the relative hardness of the stones of which the beach was composed; the exposure to, and frequency and severity of, gales; and lastly, but of great importance, the tidal range. Each case had to be judged separately on its merits.

106. For instance the whinstone and granite pebbles which occurred on the Lancashire coast had a very abrasive effect on concrete and would be expected also to have an abrasive effect on steel. Wave action could be very severe at times. On the other hand, the range of spring tides was very great, being as much as 27 ft, which meant that the grinding action of the pebbles against the sides of the steel piles in the groynes occurred only during a relatively short period of each tidal cycle. In other words, it was spread over a height of 27 ft in 12 hours on the Lancashire coast, compared with 18 ft in 12 hours at Shoreham, 6½ ft in 12 hours at Freshwater Bay in the Isle of Wight, and 14 ft in 12 hours at Sheringham in Norfolk.

107. The most severe abrasion of steel sheet-piling by wave-driven shingle of which he had a record had occurred at Freshwater Bay, where holes had appeared in the sheet-piling in the groynes within 6 years of their construction. At Sheringham, where the flints of the upper foreshore were in a constant state of movement and where the noise of the stones as they were rolled over one another and driven against the sea defences by the waves could be heard for some distance inland, the sheet-piles in the groynes at the point of maximum abrasion had been worn to about 50% of their original thickness in 10 years. At Fleetwood and the Lancashire coast, box-piles extracted from some of the groynes showed no signs of wear after about 10 years of exposure.

108. He noted that at Shoreham, with a tidal range of 18 ft, light steel sheet-piling in the Hamblin Groyne had been found to have holes in it when the west pier was being built. He assumed that this groyne was about 16 or 17 years old. It would be interesting to know the type and section of piling used, whether it had been new or second-hand when the groyne had been built, at what level relative to the beach and to high water the damaged piles had been situated, and any other information which might have a bearing on the life of steel sheet-piling under conditions such as those.

Mr F. E. Prior (Director, Messrs Peter Lind & Co. Ltd) gave further details of the constructional problems enumerated and the methods used in building the breakwaters, from the contractor's point of view.

110. Starting with the east breakwater, a heavy mobile crane was used to commence the driving of the steel piling in the curved retaining wall at the root of this breakwater. The seaward half of this wall approaching low water mark was required to be tied back to anchor blocks embedded in the beach and filling. To protect the working site they had driven a line of temporary steel joists and backed it with horizontal planking of steel-sheet piles. They had found, however, that the gravel was rapidly eroded from the back of these planks and during a series of summer storms the temporary sea defences had buckled and failed. It had been essential to change plans and provide a second line of permanent steel piles, parallel with the first to give the required anchorage. The necessary protection was completed by two temporary transverse diaphragms perpendicular to the direction of the waves and a thick mass concrete cover to prevent erosion of the shingle fill. The heavy mobile crane, which was used at first, had to be replaced by a 6-ton derrick, to give the desired radius of working from a reasonably stable support.

111. The contractors learned from their experience the desirability of giving as much, if not more, attention to constructional methods as to final design requirements for work in the areas between high and low water marks on a comparatively steep beach slope. Speaking in retrospect, it would probably have been cheaper and certainly

quicker to begin the construction of the main breakwater very much nearer the high-water mark.

112. Mr Prior then gave more details of the construction and operation of the cantilever-type construction frame shown in Fig. 6. He drew special attention to a pair of horizontal 300-ton hydraulic jacks which had been provided, one in each side girder immediately shoreward of the transverse strongback. After the completion of each foundation pour the rig had been moved 21 ft forward, the tail clamps fixed, and the rig finally adjusted and located by applying horizontal forces of up to 100 tons in each hydraulic jack. While the steel piles were being driven through the piling guide in this 21-ft length of breakwater, work had been proceeding on the form-work and the concrete of the superstructure above the previous foundations.

113. At one time they had hoped to drive at each temporary stop-end heavy steel joists down which they would slip large shutters on open-braced steel frames, the ends of such shutters to be sealed against the side piling by specially fabricated long inflatable rubber sausages; but as a result of very rough treatment by the sea in the early phases of the contract they had, probably wisely, abandoned this procedure and had driven temporary stop ends of steel sheet-piles at every move forward. On the east breakwater this had meant the driving and extraction of a 30-ft run of temporary steel sheet-piles for every 42 ft run of permanent steel sheet-piles. The wave forces had been extremely heavy at times, and they had adopted a suggestion by the Resident Engineer, Mr Elsby, and provided compression springs between the steel piles and their supporting framework, thus reducing very largely the shock effect of the breaking waves. This had proved very successful. They had employed the same device on the west breakwater, where the forces had been worse on the west side, owing to full exposure to the S.W. gales.

114. The steelwork for the east and west breakwaters had been designed and fabricated to their requirements by T. W. Palmer & Co. of Merton, London. In general, the assumed load from wave action on the steel sheet-piling in the most vulnerable position was $\frac{1}{2}$ ton/sq. ft. They had assumed a maximum internal load from concrete placed under water at the rate of $2\frac{1}{2}$ ft/hr of not greater than 250 lb/sq. ft. The total weight for the east breakwater rig, neglecting the special steelwork supporting the 5-ton crane and the kentledge, had been of the order of 60 tons. The rig for the west breakwater had been somewhat smaller, for several reasons. The overall length of the breakwater was less, and they had wished to economize on the initial installation. The work had been very vulnerable, as he had already mentioned, to storms from the south-west acting on the projecting side of the cantilever, and they had wanted to reduce the risk of reducing the length of the cantilever. The shore end of the breakwater was curved, and that led to considerable complications in designing a cantilever rig to travel round this curve. As a result of their experience, if they had to build that type of breakwater again they would employ rigs of similar basic design but with minor modifications.

115. The concreting plant for the east breakwater had consisted of a twin 1-cu.-yd mixer with weigh batcher feeding the two 6-in. concrete pumps, and they had programmed a rate of deposition through 8-in.-dia. tremie pipes of not less than 36 cu. yd/hr, so that 430 cu. yd took about 12 hours to complete. The maximum pour had been just less than 600 cu. yd. As the Author had stated, the time of commencing the pour had been adjusted having regard to the tide and weather. In general they had attempted to ensure a minimum of 10 ft of water above the concrete, whenever the upper level of this water was liable to be agitated by waves passing over the concreting "box".

116. With reference to the Author's remarks in § 33 of the Paper regarding the use of tremie concrete of 3-in. slump, Mr Prior hoped that no one would think that this had been the contractor's idea. After the mix had been made more plastic by increasing the proportion of cement from one-sixth to one-fifth by volume and by the addition of sufficient water to give a 7-8-in. slump, and using reversible 30-cwt air hoists with automatic brakes to control the tremie hoppers, he could confirm the Author's remark that very good tremie concrete indeed had been obtained.

117. Given reasonable luck with the weather and working double shifts, with increasing experience it had been possible finally to achieve a rate of construction of the foundations of the east breakwater of approximately 80 ft per calendar month, or about one complete move per week. Although on the west breakwater the rig had permitted only 13 ft stops forward compared with 21 ft on the east breakwater, the gang on the west breakwater had achieved six moves per month, or 78 ft as compared with 80 ft. The east and west breakwaters had been substantially completed by September 1956.

Mr W. H. Dunkley (Divisional Controller, South Eastern Division, Central Electricity Generating Board) said that the decision to build power stations of such capacity at Brighton had been made because the south coast was an extremely difficult area in which to find power-station sites. In each of the next 2 or 3 peak years it would be necessary to get about 900,000 tons of coal to the stations. This would mean, if 1,500-ton ships were used, that about 660 of them would have to enter the harbour in a year, so that the ordinary trade of the harbour would be seriously affected and would have a very small allowance of time in the lock. It had therefore been decided to use 4,500-ton ships, with which about 220 loads in the year would be sufficient. The Author had given the cost of the works as £2.75 million, but there were a good many other works which he had not mentioned but for which the electricity consumer had to pay. The total cost at the moment would appear to be about £3.2 million.

119. The demand for coal at power stations was extremely peaky. This was a result of the peculiar requirements of the electricity supply industry, according to which stations were built and heavily loaded and then superseded by more efficient ones which had to shoulder the base load. The full impact of nuclear power developed was yet to be felt. Shoreham Harbour had been given a wonderful entrance, improved depth and a magnificent lock, but as soon as the power stations ceased to burn coal in substantial quantities, the harbour as it had been before the works had been carried out, could perfectly well handle the coal traffic. The result would be a lopsided harbour, with a magnificent entrance and a great lock, but behind it a very small area of water with wharfs and transit sheds. On the north side, for some distance, there was a slope of about 45°, and that length of the harbour had now no commercial value. It was a curious product of the electricity supply industry—a harbour which possibly in the future would not be handling enough traffic to justify the maintenance of its depth of approach.

120. The Act provided that the electricity supply industry had to be responsible for the extra dredging of the approach channel if it wanted to bring in 4,500-ton ships. But under the Shoreham Harbour Act it would be possible for the supply industry to reduce dredging to that required to allow the lock gates to be worked.

121. Referring to § 34, it was important that pulverized-fuel ash had been used in a considerable volume of the concrete which had been placed. On the breakwater superstructures above H.W.L. about 1,200 cu. yd of concrete had been placed with the substitution for cement of 20% by volume of pulverized-fuel ash. Britain was far behind the French in making use of material which had been burnt. In the north of France it was a legal obligation on cement manufacturers to grind-in 20% by weight of pulverized-fuel ash from power stations. Recently experiments of that kind had been carried out on laboratory scale in Britain, and it was hoped to conduct them on a works scale. The laboratory tests had confirmed the French results, that the 28-day strength of a 3:1 mix made with this cement was slightly higher than when Portland cement was used. It took 6 cwt of coal to burn 1 ton of cement, so that if 20% could be burnt at no cost it represented a very considerable contribution to the national economy.

Mr J. B. Dalman (Resident Engineer, Sir William Halcrow and Partners, Consulting Engineers) said that the central mixing plant had been sited near the root of the east breakwater, from where the contractor had been able to pump concrete to the east breakwater, the inner breakwater, and the new lock.

123. Two 1-cu.-yd mixers and two 6-in. concrete-pumps had been installed and double 6-in. pipe-lines had been laid to each breakwater. No exceptional difficulties had been experienced in pumping, but towards the end of the east breakwater, where the pipe-lines had been about 1,500 ft long, it had been essential to maintain constant supervision to prevent blockages occurring. For the new lock only one concrete pipe-line was required but since the total distance pumped was nearly 1,700 ft it had been necessary to provide at the eastern end of the outer lay-by wharf, a re-mixing hopper and a 6-in. booster pump. The concrete placed in the east, west, and inner breakwaters totalled 38,000, 25,000, and 14,000 cu. yd respectively. Approximately 22,000 cu. yd had been placed at the lock. The main mass concrete had all been 6:1 by volume, with the exception of the tremie concrete, which had been 5:1. Test cube results had averaged over 3,400 lb/sq. in. at 28 days.

124. For those who had not had the opportunity of witnessing loading tests it would be worth mentioning that he had been quite surprised by the organization required for those carried out at the lock, particularly for the test on the group of four piles referred to in § 58, where the total load had been 400 tons. The load had been applied to the piles by four hydraulic jacks, which had been connected to a manifold and operated by a single pump, to ensure that the load was evenly applied. Above the jacks had been fixed two pairs of 24-in. $\times$ 7½-in. joists spanning across the 24-ft-wide north-wall trench between the cofferdam walls. Horizontal joists welded to the cofferdam piling had been capable of transmitting a total of 200 tons to the piling without risk of extracting it. It had therefore only been necessary to provide kentledge over the jacks totalling about 200 tons, but even so this had taken several days to load.

125. Two difficulties experienced by the dredging sub-contractor were of interest. A 410-h.p. oil-fired bucket dredger, having 30-cu. ft buckets and a maximum output of about 20 cu. yd/min, had been used, the spoil being dumped at sea, using one 520-cu. yd and two 400-cu. yd hopper barges and two tugs. The contractor had wished if possible to dredge by night and day, and before commencing work he had fitted a steel hood round the top tumblers of the dredger with a view to restricting the noise likely to disturb people in the neighbouring residential areas. Dredging of the outer harbour had been completed without serious complaints being raised, but it had been clear that more difficulty might be experienced in the inner harbour, where the houses were closer and where the large new power station on the southern side of the canal might act as a sound reflector. Acoustic experts had therefore been consulted regarding possible methods of further reducing the noise. It had been considered that this could be achieved only by insulating the inside of the tumbler hood, but no material could be suggested which would be resistant for any useful period both to water and also to abrasion by material projecting from the buckets. Following numerous sleepless nights during a period of onshore winds, public indignation had been aroused and the contractor had been forced to agree not to dredge between the hours of midnight and 6 a.m. during the latter half of the inner harbour dredging.

126. For about a month of the inner harbour dredging, progress had been hindered by difficulties experienced in clearing stiff clay from the barges. Special barges for handling clay could not be obtained and it had been necessary to devise methods for facilitating clearance of those available. Methods which had been tried with little or only moderate success included the use of immersion-type concrete vibrators; the use of detergents to lubricate the individual clay lumps as they fell into the barges and to reduce the surface friction over the hopper walls; and jetting with water jets. Eventually it had been found that by using water and air jets simultaneously, combined with manoeuvring the barge at speed with the doors open, more rapid dumping could be obtained.

Mr A. H. Toms (Research Assistant to the Chief Civil Engineer, British Railways, Southern Region) asked how it was that the soil survey carried out in advance of the work revealed conditions so different from those encountered at foundation level in the

lock area. He had hoped to find a drawing, in the Paper, showing the borings made. A considerable proportion of the shingle at Hove consisted of granite, quartzite, and sandstone. This presumably had drifted eastwards from Cornwall and Devon. Experience at Folkestone, Newhaven, and elsewhere tended to confirm the fact that any large obstruction such as a breakwater projecting out to sea would trap a good deal of this eastward-drifting shingle. Dredging from the harbour mouth at Newhaven might confirm whether or not any passed across harbour entrances in deep water. He believed that no shingle was recovered from that point, proving that such breakwaters were an obstruction to the drifting shingle.

128. Before the last war it had been the practice to tip about 30,000 tons of shingle annually on to the western end of the foreshore at Folkestone Warren in order to maintain a reasonably satisfactory beach. At this time the accumulation of the shingle in the eastern end of the Warren was such that one could step off the top of the sea wall straight on to it. During the war the shingle tipping could not be continued and by about 1948 there had been a loss of about 20 ft depth of shingle in front of the sea wall in the eastern end near Horsehead Point. This was due mainly to the eastward drift.

129. At Newhaven, if one threw a pebble over the side of the east breakwater it would stay where it fell, but if one dropped it into the sea not much farther east it would go right along to the groyne at the far end of Seaford beach. The conditions at Seaford and Folkestone Warren were somewhat similar: in both the natural supply of shingle appeared to have been interrupted by construction of harbour breakwaters to the west, and the middles of the bays had become seriously denuded in time.

130. He had been surprised to read in the Paper that it was expected to get a free-draining gravel layer at about the foundation level of the lock and that it should have been thought possible to construct a lock in those circumstances without having to face a considerable hydrostatic problem. The foundations were well below mean sea level, and there was ground-water coming from the land behind which implied conditions such as gave trouble both at Dover Train-ferry Dock and at Southampton Graving Dock.

Mr M. L. Wolfe-Barry (Senior Engineer, Livesey & Henderson, Consulting Engineers) asked if the Author, in replying to questions on the abrasion of sheet-piling, would also say what anti-corrosive measures he had used on the permanent sheet-piling which was exposed to sea action, how it had stood up in practice, and what maintenance had been allowed for.

The Chairman referred to the Author's remarks about the gates not being completely watertight and asked how far in fact that really mattered in the case in question, apart from the psychologically depressing effect of seeing water pouring through a gate. Very often, he said, particularly in circumstances such as those at Shoreham, it might be quite satisfactory to allow a certain amount of leakage through gates if the water-level in the dock behind was to be made up at subsequent high tide. There was a small enclosed dock at Southampton protected only by one pair of gates and not by a proper lock. If they did not lose a certain quantity through the gates at every tide it would not be possible during the period when spring tides were falling away to open the gates at the next high tide. There was a sluice provided to guard against the possibility of the gates being too water-tight. There had been no indication of the level to which the water had to be maintained in the area behind the lock at Shoreham Harbour. Had a small leakage anything more than a purely psychological disadvantage?

133. Mr Jellett would like to support very fervently Mr Wilson's suggestion that members' experiences in the administration of the "General Conditions of Contract" should be collected to provide information in connexion with any future revision of that document. A scoreboard showing which clauses caused the most argument might be very interesting, and it might be possible to classify contractors by their reading habits—those who read it before they tendered, those who read it before they made a claim, and those who waited until the engineer read it to them.

Mr P. G. K. Knight (Central Electricity Generating Board) said that the mere replacement of cement was not the most effective way of using pulverised-fuel ash in concrete. Work in progress for the Board (which had originated in Mr Dunkley's own Division) had indicated that very much better results could be obtained. It was hoped to publish some information about it in the near future.

The following contributions were received in writing.

Mr R. W. Bishop (Superintending Civil Engineer, Admiralty) noted that the design of the breakwater had involved the driving of steel-sheet piling, the upper portion primarily serving as shuttering for mass concrete. This method of construction was obviously expensive. Had the alternative use of precast concrete cellular blockwork been considered?

136. In § 33 the Author stated that for the breakwater hearting it had been found necessary to use a 5:1 concrete with a 7-in. slump. Such a concrete was rather sloppy for even ordinary work and Mr Bishop therefore wondered what strength this mix, with such a high water/cement ratio, was expected to develop under water; furthermore, was there not a danger that this concrete would segregate when pumped?

137. It had been mentioned in the discussion that a part of the bank adjacent to the Brighton road had collapsed after the canal had been dredged. Fig. 9 indicated that, at the western end of the new lock, lignite was encountered at 78-88, that is 6 ft below the final dredged level of the canal shown on Fig. 1; if the level of the lignite was constant along the length of the canal, it might have formed a surface on which a slip circle could have developed. Had this been borne out by the bore results?

138. It was generally recognized that, on an established foreshore, littoral drift usually occurred in a zigzag fashion and was most marked in shallow water. However, from observations Mr Bishop had made when he visited the Frisian Islands with the late Dr Herbert Chatley, just after the war, there seemed little doubt that littoral drift also took place in comparatively deep water; he would therefore have thought that, whereas littoral drift might temporarily cease at Shoreham, it would probably recommence after a new beach had built up along the back of the new West Breakwater.

Mr J. R. D. Francis (Reader in Fluid Mechanics, Imperial College, London) observed that there were two important questions concerning the littoral drift at Shoreham. The first was: why had shingle piled up on the west side of the old pier, whereas, according to the Author's remarks when introducing the Paper, the shingle on the west side of the new west pier was being scoured away? The second question was really two questions: how had shingle (apparently) passed the harbour mouth formed by the old piers: and was shingle now passing the new piers to replenish the beaches east of the harbour?

140. These questions could be answered if two special features of the conditions at Shoreham were considered. The first, locally well known, was that the west-going tidal stream began about 2 hours before high water and caused hydrodynamic "breakaway" from the ends of the old piers, with a consequent eddy to westward of them. Secondly, the new pier had been sited so that its offshore end was in a direction more nearly south-east and therefore more nearly in line with the west-going current, thus making breakaway far less likely.

141. The reversal of the inshore tidal current implied that, during the last 2 hours of the flood into the harbour mouth, the well-known "wind-against-tide" condition obtained to westward of the piers. This condition, when aggravated by the prevailing south-westerly winds reaching gale force, would produce rough seas, which, owing to the high sea level, would agitate the shingle on the beach far more than when the tide was running in the same direction as the wind. With lower water levels the effect was of less importance, for only the upper parts of the beaches were shingle, the lower parts being small gravel and sand.

142. Thus in the period from 2 hours before to 2 hours after high water when the water level was high and the shingle was agitated by waves, the current in the eddy to the

west of the old west piers carried shingle to the southward along the pier (see Fig. 15): at lower water levels the agitation was less or non-existent, so the direction of the tidal current was of no importance. With the new piers, there was probably no breakaway or eddy, so that despite the agitation of the shingle, there was no eddy current tending to bring it toward the pier end; there might be a small eddy on the ebb, but it was probably confined to the offshore end of the west pier, and did not extend sufficiently to the north-west to move shingle. In fact, during the whole high-water period, when the tidal current was west-going it was travelling to the north-west along the shore end of the pier and so caused a certain degree of scour immediately to the west of the new west pier, where the convergence of beach and pier caused a concentration of wave energy and consequent concentration of shingle agitation (see Fig. 15).

143. It would therefore be seen that the old harbour configuration had caused a shingle bank to accumulate near the head of the west pier in the high-water period. This bank could then be washed across (and into) the harbour entrance on a succeeding flood (east-going) tide, accompanied by a westerly gale. Thus the old harbour mouth was always silting to form a bar (which from all accounts had always been shallowest on the east side, presumably because the ebb kept the west side of the harbour clear). With the new harbour configuration there was no such bank available for movement on the flood, though possibly the shingle beach immediately to the west of the west pier might move a little to the south-east along the pier on the flood, to be replaced to its original position by the west-going stream starting 2 hours before high water. It seemed therefore that little or no shingle was likely to pass the new piers to form a bar, at least for a long period of years, until the west-to-east littoral drift along the coast increased the shingle width along the whole beach for a mile or more to the west of Shoreham Harbour. Such a situation seemed now to be in a more advanced state at Newhaven where the west breakwater was of a shape somewhat similar to that of the new west pier at Shoreham.

144. In fact, the tidal currents around the new west pier head in the period from 2 hours before high water to high water, and also during the first 4 hours of the ebb, now acted in a way to create scour: before high water the action was like that of a bridge pier in a river, creating a pot-hole slightly upstream of it; and on the ebb the outflow from the river turned sharply round the pier head to the west with a corresponding scour. On the first 4 hours of the flood with an east-going current, the inflow to the harbour would cut sharply around the west pier head, again scouring. The Author had mentioned that this scouring had been observed already.

145. The beaches immediately to the east of the harbour entrance might not be noticeably eroded for some time, since the agitation of the shingle during the important period over high water would not be so intense in a westerly gale as the agitation of the shingle to the west of the harbour, because of the shelter of the new piers. Thus the *shingle* beach might not be moved although sand might be moved in front of it. Since there would, no doubt, be a breakaway from the end of the west pier on an east-going tide, there would be an eddy off the east beaches, giving a slight west-going current there even if the tide in the offing was east-going. There would be of course a west-going current there when the tide was going to the west, so there was a possibility of a slight accretion of sand on these beaches immediately to the east of the harbour entrance, receiving sand from a point further to the east. Such accretion had certainly occurred with the old piers.

146. The worst erosion therefore seemed to be likely to occur at a point well to the east of the harbour entrance where the normal littoral drift took shingle to the east, and the local eddy took sand to the west as mentioned above. How far this point was to the east must be a matter of guesswork, but the extent of an eddy due to breakaway might well be about 3 or 4 times the projection of the object causing it. Thus a point perhaps 1,500–2,000 yd east of the 1,500 ft long pier should be watched. Such a situation had arisen at Newhaven, 12 miles east of Shoreham, where there was a local accretion of sand in a similar situation, but further to the east, at Seaford, there was a large erosion of shingle.

147. Although it was dangerous to argue from a particular example to a general case,

FIG. 15.—DIRECTIONS OF TIDAL CURRENTS NEAR THE ENTRANCE TO SHOREHAM HARBOUR, AT DIFFERENT TIMES RELATIVE TO HIGH WATER. The upper five diagrams are for the old piers, now demolished, and the lower five for the new piers. Flow directions are partly observed, and partly deduced. Dotted lines show approximate positions of water lines on beaches. Spring tides are assumed.

it might be interesting to speculate generally on the connexions between littoral drift and reversals of tidal streams at times other than high water or low water in conjunction with prevailing gales. This was particularly relevant in view of the possible success of the above account which described the actions at Shoreham without invoking any question of shingle movements in great depths of water or arriving from offshore areas. Around British coasts, tidal reversals were not uncommon at times other than high or low water, and it would be interesting to inquire if the reversals of tidal currents to oppose prevailing gales in the all-critical high-water period (say 2 hours each side), were also accompanied by noticeable littoral drifts.

Mr J. R. A. Garland (Chief Engineer, West Sussex River Board) asked to what extent did the model (§ 8) reveal the probability of erosion which had occurred on the foreshore at Kingston? Were the spending beaches less efficient than had been hoped?

149. When introducing the Paper the Author had referred to the unanticipated scour which had occurred immediately west of the west breakwater. This would appear to be due partly to the reflexion of waves from the south-west and the angle of the breakwater. It was local in extent and there was some accretion in the beaches farther west.

150. The Author had suggested that the tendency for bars to form just inside the harbour mouth might be due to the River Adur depositing its silt load when it met the sea. The Adur did carry a lot of silt under flood conditions, but there was a considerable variation in channel section between Old Shoreham Bridge and the harbour mouth with corresponding variations in velocities and Mr Garland would have expected the deposition of silt to have occurred before the mouth was reached.

Captain A. G. Stephenson (General Manager, Shoreham Harbour Trustees) referred to the failure of the steel sheet-piling of the Hamblin Groyne (§ 28) and gave some details of the piling used in the original construction of the groyne, which had been completed in November 1945. Consisting of Frodingham No. 2 steel piling, it had been built to prevent shingle passing round the head of the west pier and causing shoals in the harbour entrance. The foreshore on each side of the groyne consisted mainly of shingle, although some sand was mixed with it, particularly on the lower part of the beach.

152. In March 1953 it had been observed that over a length of 50 ft on the east side of the groyne the webs of about twenty steel sheet-piles had been worn right through by the abrasive action of the shingle. The wear had taken place in the region between the then existing beach level and a line about 2 ft above it and also between low water mark and approximately half tide level. In the case of the damaged piles nearest low water it was the webs facing north (towards the land) which had been worn through. Whereas in those farthest from low water it was the webs facing seaward which had been most heavily eroded.

153. The beach level on the east side of the groyne was lower than it had been when the groyne was built and so some of the shingle beneath the base of the concrete inside the groyne had been scoured away by wave action.

154. The Consulting Engineers for the construction of the groyne had been informed of the premature wear in the steel-piling and had been asked for their opinion on the matter. Their reply had stated that the construction of the groyne had been carried out as an emergency measure during the war to give protection to the harbour entrance. Permission for construction could only be obtained on the basis that the cost was to be kept to the minimum, and so the groyne had been designed as a mass concrete gravity wall bearing on shingle. It had been specified that the level of the shingle be maintained at least 1 ft above the foundation level of the concrete wall in order to prevent the undermining of the wall by withdrawal of shingle under the concrete. It had been further stated that the sheet-piling used as shuttering was surplus piling from other work; none other was available and it was not the type best suited to the work. The Trustees had been asked to appreciate that the design of the wall had been determined by the circumstances prevailing at the time.

155. The west side of the groyne was fed by the littoral drift and so no difficulty had

been experienced in ensuring that the level of the shingle on that side was always above the base of the concrete. However, the variation in beach level caused by wave action on the east side of the groyne was such as to render it impossible to maintain the beach level on that side above the base of the concrete—even on those occasions when the shingle between the groyne and the west pier had accumulated to such an extent that it was passing round the head of the pier into the harbour entrance.

156. Captain Stephenson had not been in the service of the Trustees at that time but he assumed that the sheet piles used for the Hamblin Groyne, although not new, were in fairly good condition. He knew that the steel sheet-piling used for the permanent works of the Brighton Power Station was Larssen No. 3 and it seemed likely that the piling incorporated in the Hamblin Groyne might have been used as temporary works for the power station outfall.

The Author, in reply, said that the model which had been built prior to the execution of the work was a fixed-bed model, and no attempt had been made to adapt it to determine in what way the existing coastal regime would be affected by the construction of the breakwaters. The purpose of the model had been to demonstrate what disposition of the breakwaters would provide the best conditions inside the harbour, and the model results had substantially been borne out in practice.

158. A small surge in the Eastern Arm had also been detected on the model but had been discounted as "model effect". Nevertheless, a surge of about 18 in. amplitude was actually being experienced and attempts were being made to alter the resonance of that part of the harbour by varying beach profiles and removing redundant structures. Erosion of Kingston Beach had always been expected but no action was being taken to stabilize it until this was shown in practice to be essential.

159. Suggestions had been made that the inadequacy of site investigations might have led to the foundation difficulties subsequently experienced at the lock. More than twenty boreholes had been put down, five of which had been within 100 yd of the new lock. The layers of lignitic clay had been disclosed but not the underlying thin layers of softer clay; nor could the behaviour of the latter under artesian pressure when stripped of their overburden be determined from the borings. Owing to the oscillation of the point of discharge of the River Adur over the years, interpolation of the levels of the various substrata could not be positive. This applied also to the area adjacent to the slip on the north side of the Canal.

160. As was mentioned in §§ 42 and 44, the hydrostatic problem at the lock had been foreseen. Had gravel beds been encountered as anticipated it would have been possible to drain through them, even under pressure.

161. The Paper had perhaps given the erroneous impression that the whole of the work had been brought to a standstill while the lock walls were being redesigned. That was not so. Only the work on the lock had been halted, although some other works had been adversely affected by delays at the lock. For instance, permanent sheet piling for eventual use on other sections of the site, which had been used in temporary works at the lock, could not be released as anticipated. There had been other effects arising out of the delay but in the main it was not difficult to allocate financial responsibility. The Conditions of Contract used were basically those agreed by the Institution, the Federation, and the Association, and Clause 12 excluded the Contractor from liability in the event of conditions which an experienced contractor could not have foreseen. The only real difficulties had been in determining just how far the effects extended and in assessing their monetary value.

162. The value to others of a cumulative record of contractual settlements was doubtful. First, because the particular conditions under which a scheme was carried out had a marked bearing on the assessment, and secondly, because the final stages of a settlement were invariably achieved by some relaxation of contract obligations and entitlements by the employer and contractor respectively. Of much more value would be the publication of the results of arbitrations arising out of the Standard Conditions of Contract, or failing that a paper or debate on the administration of the Standard

Conditions. It was probable, however, that neither the engineer nor the contractor would enter into such an arena with complete frankness, since that would inevitably disclose the weaknesses in their armour.

163. The several questions asked about the lock could conveniently be grouped together. Various types of main discharge culvert ports had been tested on the small lock model and discharge streams had been directed in various combinations of direction. In all cases where the streams were directly opposed across the apron the arrangement had been found unstable; eventually one stream had swung towards the gates and the other towards the front of the apron. The arrangement decided upon not only gave the best flow over the leading edge of the apron but caused a minimum of erosion downstream of the apron. Soundings and underwater inspections had been carried out and no subsequent work on the bed had been necessary.

164. The lead-in jetties had purposely been made as massive as possible, because ships' captains were always more careful when approaching structures which might damage their ships. Mr Wilson had probably misunderstood the Author's introductory remarks, which had included a reference to jetties with concrete decks on steel piles—not timber piles as mentioned by Mr Wilson.

165. Oil-hydraulic gate-operating mechanism had been selected in preference to electro-mechanical because of its economy and its minimum of moving parts. Gates in each lock could be overhauled by restricting locking times to high water or directing all traffic through the other lock. This could mean, in the latter case, a reversion for a short time to the transportation of all coal in 1,500-ton colliers.

166. The Chairman's remarks about the leakages through lock gates were perfectly reasonable and there was no doubt that leakage could be and was tolerated in many cases. It would, however, be extremely difficult to persuade a client that he had nothing to worry about when he saw a fair-sized leak past a new set of gates. He most certainly required them to be as near perfect as possible on acceptance even if they did sustain damage from cables, fenders, and vessels after a very short time.

167. The Author agreed that the use of pulverized fuel ash could usefully be extended, and he would be interested to read in due course the publication referred to by Mr Knight.

168. There was no doubt that the Contractors' work on the main breakwater would have been facilitated if the deep-water type of design had been started close in-shore; but it was doubtful whether the Electricity Authority would have had the benefit of any resulting economies, since the Contractor became familiar with the difficulties only when he was trying to build the breakwater—not at the time of tendering.

169. The use of cellular blockwork for breakwater construction had been considered in the design stages, but bed preparation difficulty in the unpredictable weather conditions had been a strong argument against it.

170. The very high slump had not produced segregated aggregate; in fact, the lower the slump the more segregation there had seemed to be in the pumped, tremmed concrete. It had been possible to examine concrete between tides when the temporary transverse diaphragms were removed, and also in a special inspection pit which had been formed in the breakwater to a level below low water. Cubes cut from the between-tides portion of the breakwater had shown strengths up to 2,990 lb/sq. in. at 40 days.

171. The erosion west of the west breakwater, where an accumulation of shingle would normally have been expected, was extending westward. It was agreed that the beaches even farther west were now quite full.

172. Mr Francis had put forward an interesting theory that a hydrodynamic break-away in the west-going tide was responsible. The Author had no doubt that something of that nature was probably contributory, but he had found it possible by a re-orientation of the tidal streams to produce (on paper) much less eddy and, with a little licence, to reverse its direction. Others had since suggested that deep-sea waves from the south-west were being reflected by the west breakwater in a north-westerly direction, which again might be contributory. The Author regretted that it had not been possible to make a careful and detailed study of this particular case, which he was sure could help

in the appreciation of the littoral drift problem. He was unable to indicate at which period of the tidal cycle accretion or drift was most pronounced.

173. The Author confirmed that during the committee stages of the Bill enquiries had been made regarding the type of material dredged at Newhaven, but there had been no confirmation that any worth-while quantity of shingle had been recovered. On the other hand it had been separately confirmed that a "foreign" shingle deposited artificially west of the west breakwater at Newhaven had later been found in small quantities on the Seaford beaches to the east.

174. There was little doubt that in-shore eddies were responsible for the results obtained by Mr Toms when he had dropped pebbles from the Newhaven breakwaters.

175. One or two members had enquired about the behaviour and life of the steel sheet piling used on the Hamblin Groyne and the Author could not do better than refer them to the contribution by the General Manager of the Shoreham Harbour Trustees, received subsequent to the discussion.

176. Mr Wolfe-Barry had enquired whether any measures had been taken to protect the piling on the new breakwaters from similar wear. Although the structures did not depend for their stability on the sheet piling, it had been decided, in certain places, to bolt 9 in. $\times$ 9 in. vertical, and 9 in. $\times$ 3 in. horizontal timber cladding to the piling in the inter-tidal zone on the east breakwater. Wear had not been unreasonable, and maintenance was roughly the same as on normal beach defence systems.

177. In conclusion the Author recorded the ready assistance and co-operation given by Messrs Rendel, Palmer & Tritton, Consulting Engineers, who had been retained throughout the design period and contract by the Shoreham Harbour Trustees to watch their interests.