

to the lock wall is of a solid construction, utilizing the concrete blocks from the invert kentledge.

On the south side of the entrance the original bullnose had been specially strengthened to receive vessels, and to enable them to turn on the bullnose for approach to the entrance. The bullnose has been demolished and an alternative method adopted using a flexible fender, 120 ft long, formed with high-tensile-steel piling at 3-ft centres, surmounted by a continuous reinforced concrete capping beam. These works are not yet completed.

ELECTRICAL

A diagram (not published) which accompanies the Paper shows general electrical arrangement for the entrance.

The provision of a new high-tension intake and the conversion of this area to 3-phase A.C. supply have enabled a very satisfactory arrangement of electrical cabling to be undertaken. Safeguards have also been incorporated to prevent the possibility of sluices being opened until the gates are closed. In addition, duplicate control has been provided at the south side of the lock from which a pair of gate machines can be controlled simultaneously.

New lighting standards, at about 85-ft centres and 25 ft from the coping, are provided with special dispersive reflecting lanterns.

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The Paper is presented by permission of Mr G. A. Wilson, M.Eng., M.I.C.E., M.I.Mech.E., Chief Engineer, Port of London Authority, to whose designs the work described was undertaken.

The principal contractors were Messrs John Mowlem & Co. Ltd, with Soil Mechanics Ltd as sub-contractors for the ground-water-lowering system, and Messrs Head Wrightson & Co. Ltd, sub-contractors for the gates and machinery.

The Paper, which was received on the 30th September, is accompanied by eight photographs and fourteen sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Discussion

The Author introduced the Paper with the aid of a film.

Mr J. R. Malcolm (Senior Engineer and Agent, John Mowlem & Co. Ltd) observed that from the contractor's point of view there had been many difficulties and problems to solve, and amongst them were the construction of the two dams and the best method of constructing the knees. The dams had required careful consideration and especially the outer dam, where, owing to the presence of a scour hole about 20 ft deep in the chalk, it had been difficult to provide a satisfactory foundation for the blockwork dam originally proposed. After some consideration it was decided to abandon the idea of a blockwork dam and to use a steel sheet-pile dam, as shown in the Paper. Both the dams had proved to be remarkably tight; in the case of the outer dam practically no pumping was necessary at low water, and at high water an 8-in. pump was able to deal with the flow. At the inner dam a 6-in. pump was adequate.

To construct the reinforced concrete knees at the foot of the existing walls it had been necessary to make pockets in the invert 11 ft wide at 55-ft centres. The sides had to be cut plumb and true to dimension to allow the reinforcement to be placed accurately so as to give the correct spacing between the bars in adjacent sections and also to allow the concrete to be poured for the whole 11-ft width of the section, so as to avoid the use of shuttering or excessive trimming of the concrete when the adjacent section was excavated. After some trial it had been found that to cut a chase round the three sides of the pocket gave the best results, using a Siskol mining cutter. Another advantage of using the Siskol cutter in that way was that when blasting took place there was a free face on three sides of the area, which stopped the shock going to the rest of the work.

Mr Malcolm illustrated his remarks with a series of lantern slides. The first showed one of the pockets and the straight face obtained by using the Siskol cutter, also the various steps which had to be cut in the bottom, which required careful work.

The Siskol cutter was secured to a 10-ton kentledge block, which provided a steady base and enabled an accurate cut to be made. The block was handled into position by a 15-ton derrick. The cutter could be swung in a vertical plane about an axis and at the same time raised and lowered to control the depth of cut. Various lengths of boring bar were available depending on the depth of cut to be made. The cutter head which was most suitable had five chisel-shaped prongs. The cuttings were removed by means of a rake.

Timber shuttering was used for the curved parts of the knees. It was handled as a unit, placed on previously levelled packings and weighted down with eight 2-ton kentledge blocks.

The quality of the concrete in the old walls varied considerably and in some places a good deal of it had to be cut out. Pockets were cut in the face of the walls to give a key for the reinforced concrete facing, in which B.R.C. fabric was used. The shuttering used was of timber faced with plywood in panels 5 ft 6 in. long and 3 ft 6 in. high, and a large number of uses was obtained from those without trouble.

At each end of the lock emergency sills and hollow quoins were provided and the latter were formed in the concrete facing of the wall. To make them as true as possible without dressing after the concrete was placed a quoin shutter was made in one length to the full height of the wall. It was made of timber faced with plywood and assembled on a No. 5 Larssen pile. The shutter was accurately located at the bottom and bolted back to the wall by 1½-in.-dia. bolts. At each side of the quoin shutter climbing shutters were arranged which brought up the lifts of concrete as required.

As mentioned in the Paper, most of the granite work in the lower part was quite good. There had been an attempt to repair one of the old sills under water by cutting back the granite, inserting steel bolts, and securing a reinforced concrete patch, but the patch had not held.

The gates and sluices were tested with a head of 40 ft of water on the pressure side only, and all the gates and sluices were satisfactory at the first test.

The work had normally to be carried on until 6.15 each evening, so that during the winter lighting was necessary. A system of sodium lighting had been provided, which had given excellent results.

Mr F. H. Allen (Assistant Director, Hydraulics Research Station, D.S.I.R.) remarked that he had been concerned with the model investigation (described briefly on p. 154), which had been undertaken to find out whether or not the quite considerable scour which was known to have taken place on the riverward side of the apron could be reduced by relatively small alterations to the sluice outlets. The experiments had been carried out by Mr W. A. Price and the Port of London Authority's model research staff, under the direction of the Hydraulics Research Station of D.S.I.R.

Mr Allen showed several slides to illustrate his remarks. The model was constructed to a scale of 1:20 vertically and horizontally. The bed of the riverward section had been moulded in sand which had a mean diameter of about 0.2 mm. The results obtained from tests with one design of sluice had been compared with those obtained from tests on various

revised designs; the relative merits of the designs had thus been compared, but no attempt had been made to draw any conclusions about the actual amount of scour. Since the model had been constructed in 3 weeks and the experiments carried out in 6 weeks there had been relatively little time for an academic research investigation. The model had been operated under the most severe conditions, i.e., the conditions liable to cause the greatest scour. That had been done by maintaining high-water impounded dock level in the lock and L.W.O.S.T. level in the river, and allowing the water to discharge continuously under those conditions through the sluices. The water was circulated in the model by means of a system of pumps, pipes, and constant-head weirs.

Each main sluice was originally of constant width throughout, and the outlets or sluiceways branched sideways from it. When that old design was tested in the model, it had been found that by far the greatest part of the discharge passed through the final or riverward sluiceway; and the Author's film, taken when the lock had been filled again after the completion of the works, showed that that was borne out in nature. A small amount of water discharged through the next sluiceway and very little through the others.

The aim had been, therefore, to equalize the flows from the sluiceways and thus to equalize and reduce the jet velocities, so avoiding the highly turbulent conditions which were the outcome of the original design. That had been done in the first place by reducing the width of the main sluice in five roughly equal steps; at the same time the entrance to each of the sluiceways from the sluice was formed as a slight scoop, so that part of the water was encouraged to flow into each sluiceway. In addition the outlets were flared. That produced very encouraging results, but it transpired that there was some difficulty in reproducing the flared outlets on the actual structure because of the reinforcement of the concrete. A revised design (B) had therefore been tested in which the flared outlet was used only for the riverward sluiceway and the others were left unaltered. The reduction in the width of the sluice was as in the first revised design (A). The third revised design (C) was an attempt to equalize the discharges still further by slightly altering the reductions in width of the main sluice, but that had not been altogether successful.

The model was run for $9\frac{1}{2}$ hours with the existing design of sluice, and the scour hole which developed was very similar in depth and shape to that formed in nature. With design A two scour holes developed. The maximum depth of scour, measured after an experiment of the same duration, was relatively small, being in the region of 8 ft as against $18\frac{1}{2}$ ft. That design had not been adopted, partly for structural reasons, and partly because there was some slight disturbance and scour at the edge, which it was thought might possibly cause some damage to the wing walls. Design B showed a return to the development of one scour hole, with a maximum depth of only 10 ft, as against $18\frac{1}{2}$ ft, and no scour at the edges. Design C was similar, but the depth of the scour was a little greater. Design B had therefore been recommended and adopted.

Mr G. E. Wild remarked that the lowering of the ground water had been an important feature of the works. The system started working in July 1953. There had been a short pause while the abutments of the outer dam had been made watertight and then it restarted in September 1953, and ran continuously until Christmas Eve, 1955, i.e., about $2\frac{1}{4}$ years. During that time it had been pumping water at the rate of about 2,000 g.p.m. It started off as 2,500 g.p.m. but, as expected, the discharge decreased in the first 2 or 3 months and thereafter a discharge of about 2,000 g.p.m. had been maintained consistently. That was extremely close to the predicted amount, which was pleasantly surprising in view of the fact that in calculating the quantity to be pumped it was necessary to select a permeability for the ground on the basis of past experience, because in Thames gravel the permeability locally could vary enormously and laboratory tests for permeability on such soils were not of much avail. A good deal of prior information, however, had been available from other jobs. It has also been necessary to decide whether any allowance ought to be made for the fact that the river and the lock, sources of water at a constant head, were well within the radius of influence of the wells. The decision arrived at had been that they were probably so silted up that they would not greatly affect it, and that proved to be the case.

If it was necessary to pump 2,000 g.p.m., the question arose of whether or not twenty wells were really needed. Mr Wild showed a slide representing longitudinal sections through the two lines of deep wells on the north and south of the lock. When the matter was being considered at the tender stage, borings were available only at the east and west ends of the lock and there the chalk underlying the gravel was considerably higher. In the boring of the wells it was discovered that there was a much lower area in the middle. Had that been known at the start, a small number of very much larger sumps might have been a satisfactory alternative. That had in fact been considered in the early stages of design, but the use of sumps as opposed to deep wells had been discounted, because, if it was necessary to get the water down very close to the chalk, in order to provide a sufficiently large wetted perimeter the sumps would have to be very big, and the cost of sinking them without losing any ground at all behind the old lock walls would have been very high. It would also have been necessary to take special precautions to prevent the corrosion of the sump materials, which might have caused a subsequent collapse.

Once it had been decided to use bored wells of a size where only one pump could be installed in each well, and where in consequence the well was out of action whenever the pump had to be cleaned, it was not just a matter of dividing the anticipated quantity by the capacity of the pumps; it was necessary to allow for wells being out of action, and also for a great deal of variation between the yields of individual wells. With some of the wells when pumping was first started there were yields of 40 g.p.m. per foot of immersed depth, whereas with others the figure was less than 6 g.p.m. After about $2\frac{1}{2}$ years of pumping the yields had about halved. That arose from the deposition of oxide in the filter gravel round the actual wells. Mr Wild considered that the number of wells used was just right, with the exception of the outer dam and the inner dam.

With regard to the standby arrangements for electrical supply, the Author mentioned that generators had been provided which were capable of running the whole supply, and those were started manually by operators who were on constant duty. That was possible because it was not a full artesian case. It had in fact been calculated that in the bulk of the lock it would take about 5 hours for a dangerous pressure to reassert itself on the base. At the riverward end, however, where the chalk was very high and the silt was a little lower, there was a full artesian case and it had been expected that the pressure would reassert itself very quickly. That had in fact proved to be the case when the pumps had been stopped and readings taken on the recovery of pressure. It had taken about 8 hours for the pressure to reassert itself in the centre of the lock, but at the riverward end the pressure had in fact reasserted itself within half an hour. Had it been a full artesian case, the pressure might be fully re-established in a matter of minutes. That had occurred on previous similar jobs, and there it had been necessary to have fully automatic changeover from the main supply to the standby generators.

Mr W. J. Sivewright (Assistant Engineer (Civil), Port of Bristol Authority) said that he proposed to confine his remarks to the alterations to the sluices and the question of the scour hole, with particular reference to similar work which had recently been carried out by the Port of Bristol Authority. At the 100-ft entrance to the Royal Edward Dock at Avonmouth a scour hole 100 ft across and 35 ft deep below bed level had developed since 1908. In the case of the Royal Edward entrance the apron consisted of monoliths 23 ft deep and founded 4 to 5 ft in red keuper marl. Above that there was merely soft silt. The sluice culverts were about 8 ft wide and 11 ft 3 in. high, each with two openings into the outside channel, their cross-sections being 6 ft wide and 9 ft high.

The scour had extended so far that the red marl under the monoliths was beginning to scour away, and, although the undermining of the monoliths was not so extensive as to render them unstable, the state of affairs which prevailed could not be allowed to continue indefinitely. Consequently a model, largely constructed of timber, had been made in the Authority's workshops, primarily to determine what material, if any, could be used to fill the hole and prevent further undermining, and also to investigate what other measures could be taken to minimize future scour.

The model in question was 1:50 true to scale, which suited the available room for erecting it. As in the case of the Gallions model, water was circulated by means of a pump. Only a short length of the lock above the gates had been reproduced, and a series of baffles was used to still the water before it passed through the model sluices. The pump discharged into the lock and an adjustable weir maintained the level of water behind the lock gates. The surplus water drained back to the storage tank. The model extended between the solid entrance piers for a distance of five lock widths beyond the length of the apron. The level of the water in the entrance was controlled by an adjustable weir, after which the water passed over a rectangular measuring notch before coming back to the storage tank for recirculation.

What was the rate of flow from the Port of London Authority sluices and how was the actual flow measured? The flow adopted in the Gallions model was presumably to the $2\frac{1}{2}$ power of the scale and it had been stated that the model was operated with high water inside and low water outside. In the case of the Avonmouth entrance, the discharge from the actual lock had been measured by timing the water level past the gauge marks on the lock wall with the sluices wide open, and it had been surprising to find that for equal decrements of water level the increments of time varied considerably. The plotted figures of discharge against time showed a rhythmic variation, evidently due to a surge of water up and down the lock; but when plotted for the first 15 ft of discharge the graph of discharge against time varied very little from the best parabola through the plotted points and very little from a straight line through the points. The flow was 2,200,000 g.p.m. with normal dock water level inside the gates and mean water low springs outside, giving a head of 41.5 ft. That had been taken as the basis for the model, for the same reason as in the Gallions model, that presumably most damage was done with the greatest difference of head.

As a first assumption the same coefficient of discharge was taken for the model and for the actual sluiceways, and the actual flow divided by 17,700 corresponded in the model to a 1-in. depth over the measuring notch. With the water in the model corresponding to normal dock level the depth over the notch was 1.1 in. This difference was accounted for by the difference in coefficient of discharge for the model and for the actual sluiceways. That was considered sufficiently accurate having regard to the measurements in the model and lock. Qualitative observations on the character of flows from the actual lock and in the model were made and confirmed the general accuracy of the model.

The bed material outside the model gates was estuarial sand with a grading of 70% passing No. 7 sieve and 1% passing No. 52, with various intermediate gradings. The model started with a bed of sand at a level equivalent to the general level of the bed of the entrance and was run until equilibrium was reached, in about half an hour. The scour hole then closely resembled the actual one and the general topography of the entrance bed was closely reproduced. There was a tendency for the coarser material to be deposited on the outside edge of the scour hole. That was removed by hand, which was perhaps cheating, but this procedure was considered justified since if the material had been finer it would have been transported farther down the model. Model concrete cubes representing cubes from 1-ft to 7-ft side were all swept away from the apron with the sluices running full open.

A long series of experiments had been carried out to examine the effect of filling the hole with rough stones of various sizes and with sand. Other experiments had been done with various water levels in the entrance and also with Rehbok teeth on the edge of the apron, but those had no effect, since the maximum size of the teeth was limited to the 2 ft difference between the apron and sill and was so small as to be ineffective.

Mr Sivewright noted that the alteration of the culverts in the Gallions entrance improved the flow. Had those alterations altered the total discharge from the sluices and altered the normal time for locking? At Avonmouth, as at Gallions, most of the water discharged from the downstream outlet, and both in the actual lock and in the model water entered the upstream outlet at the commencement of sluicing. Experiments had been carried out using diverters of various shapes, but without appreciable effect. The alteration of the main sluiceway, as had been done by the Port of London Authority, had not been

considered, nor was it practicable. It was worth observing that the inlet end of the Avonmouth sluiceway culvert approximated in shape to the outlet end of the Gallions culvert as reconstructed.

What was the nature of the bed beyond the end of the apron at the reconstructed Gallions entrance? Presumably the base of the rubble dam had been left in position.

The main conclusion from the Avonmouth experiment was that the velocity of discharge should be reduced. That might seem self-evident, but the question of locking time came in. The reduction of velocity achieved by permanently throttling the sluices to three-quarters open, thereby reducing the velocity to 50%, had made relatively little difference to the locking time. It was also concluded that 2-ft-cube stones in the hole would stay in place if the level of the stone was sufficiently below the top of the apron. There was no point in filling the hole since the Port Authority was concerned only with protecting the monolith bases.

Mr Sivewright concluded his remarks by showing several slides of the model.

Mr A. L. Reed (Technical Manager, Dock & Harbour Equipment Dept, Head, Wrightson & Co. Ltd) commented on the Author's statement that the gates were fully assembled at site in the lock bottom, all the material being supplied shaped and fabricated from the works, and explained the method of assembly. The gates had been built in the vertical position on a timber raft placed adjacent to and in line with the quoins. The raft was in two parts, one on top of the other, separated by 9-in. $\times$ 3-in. timbers suitably spaced for the eventual placing of the rollers for rolling the gates into position. The bottom deck was placed on the raft and the diaphragms were tack-welded into position. The first strake of the flat skin plating was tack-welded into position. The decks and diaphragms were assembled and tack-welded up to the level of the first butt on the flat skin plating. The centre-lines of all plates were marked and the corresponding contact points of centre-lines of contacting plates were also marked. That was essential for correct assembly and helped considerably in checking.

The first curved skin was then brought up to the four decks and diaphragms previously assembled. That curved skin had not been preformed in the works but was pulled into shape by Yale "pull-lifts." At that stage the rigid squaring frame shown in the Author's film was brought up to the flat skin plating. That rigid frame consisted of joists and channels and served as a support to the flat skin plating during further erection. It proved exceedingly useful for checking the alignment against twist and distortion during the remainder of the assembly.

The remaining deck, diaphragms, and skins were assembled in a similar manner, except that the flat skin was assembled first and a sealing run put in before the plate was tack-welded to the diaphragms and decks. Similarly, with the curved skin the sealing run was put in before the plate was fastened to its diaphragms and decks, the reason being that there was a certain amount of preliminary shrinkage and that was a means of minimizing distortion. After all the decks and skins had been assembled the heel- and mitre-post plates were brought into position and tacked in position, tacking to the flat skin first. The gate had now been fully assembled, but was only tacked except for the sealing runs in the butt welds.

The next stage was to put sealing runs in the vertical positions on all diaphragms and sealing runs on the downhand and overhead welding inside the gate, and then complete those welds with the exception of the access trunks. Four runs were then put on the outside of the butt welds and chipped back on the inside, and the inside welding completed before going outside and finishing the butts. Next the outside welding on the heel and mitre posts was completed—one welder working on the flat side and one on the curved side. The access trunks were welded last; it had been felt that to weld them as the gate was proceeding would cause another obstruction to the natural distortion of the gate. Four welders were continuously working while the final welding was taking place. The procedure was that they started in the centre of the gate and worked out towards the heel and mitre posts, and up and down from the centre towards the top and bottom of the gate.

At all stages of assembly and while welding was in progress it was essential that all plumb lines and levels should be checked twice daily, on commencement of work in the morning and on commencement of work in the afternoon. That gave a chance of removing distortion before it was permanently welded into the gate. The steelwork was approximately $\frac{1}{8}$ in. larger than was really required, in order to allow for shrinkage during the welding of the gate.

On completion of the steelwork the greenheart had been fitted and dressed to dimensions taken from the finished standing sills and quoins. The gates were then rolled into position, as shown in the Author's film, and lowered in stages by jacks on to the pintles and finally connected up to new top anchorages and the operating machinery. The three pairs of gates had now been completed, and they had been tested with a full head on the pressure side and dry on the sill side. The inner gates had negligible leaks, but it had been decided to make the other gates approximately 0.010 in. longer than the inner gates, and on test those proved to be perfectly watertight.

With regard to the machines, only temporary electric wiring had so far been provided, but that apparently worked very well. The permanent wiring would not be connected up for about 2 months, since interlocking with the sluice machinery was necessary.

Mr J. A. Dempsey (Assistant Site Engineer, John Mowlem & Co. Ltd) said that in the past 40 years only three other dams ^{2, 3} had been constructed of precast concrete blocks in a manner similar to the inner dam described by the Author, and therefore certain techniques and difficulties which arose during construction were worthy of mention. The dam had been designed as a simple gravity section founded on the natural gravel, capable of withstanding a horizontal thrust of approximately 32 tons/ft run. Under the gravel, which was about 6 ft deep, was the chalk. Both those strata were adequate to support the mass of the dam, but their ability to take the horizontal thrust had to be the subject of careful investigation. Shear-box tests had been carried out on uncompacted gravel in contact with a rough concrete surface and it was found that a movement of approximately 0.8 in. was to be expected before the full frictional resistance of the gravel was mobilized. That corresponded to a factor of safety of about 1.4. That movement under the full hydrostatic pressure would seriously endanger the watertightness of a clay seal placed between the cut-off and the face of the dam, and, once that seal had been broken, there would be a serious increase in the uplift pressure under the dam. The factor of safety against sliding would then be reduced to 1.05. Neither of those two factors of safety was acceptable in practice, especially in view of the risks to the whole dock system should any failure occur. Moreover there would be a risk of piping in the gravel should seepage flow become at all large. Therefore, it had been decided to install a row of deep wells between the sheet-pile cut-off and the face of the blockwork with a view to intercepting any seepage water which might come through the cut-off and eliminating the danger of a break in the clay seal at the face of the dam.

Furthermore, the Port of London Authority proposed to extend the existing sill a further 26 ft, to within 15 ft of the toe of the dam. To carry out the construction of that sill would have meant excavating to a depth of 5 ft below the base of the dam. They proposed that the work should be carried out in the dry. Such an operation would have been difficult and might have seriously affected the stability of the dam. The contractors proposed, therefore, that the new sill should be constructed at the same time as the dam, and under water,⁴ and that it should be extended right up to the toe of the dam, so that it would act as a strut between the existing sill and the toe capable of carrying the full horizontal thrust. That modification increased the factor of safety of the foundation as a whole to 7, and eliminated all risk of sliding and the problems associated with it.

² P. W. Bertlin, "The Construction of the New Entrance to Tranmere Dock." *Min. Proc. Instn Civ. Engrs*, vol. 221, p. 241 (1925-26, Pt 1).

³ George Ellson, "Dover Train-Ferry Dock." *J. Instn Civ. Engrs*, vol. 7, p. 223 (Dec. 1937).

⁴ See footnote 1, p. 143.

The deep wells which he had mentioned were left in the design purely as a precaution, mainly with a view to reducing any uplift pressure under the new sill caused by the seepage water. In the event, those deep wells produced so small a yield that several were abandoned at the start.

The other point which he wished to mention was the sealing of the face of the dam. There the joints between all the blocks in the water face of the dam had a recess cast round the face of each block, and that was filled by a canvas sleeve placed by the divers under water. The canvas sleeve was then pumped full of grout until it entirely filled the recess, thereby closing the horizontal and vertical joints between the blockwork.

Once that sealing was complete the dam was tested under the water load. It was when they had approximately half the full differential head on the dam that considerable horizontal movement took place between six of the courses of blockwork, and the seal in the face of the dam was broken. That horizontal movement had been arrested only by interlocking recesses cast on the top and bottom faces of all the blocks. A survey had been carried out as soon as that movement was noticed and showed that a displacement of approximately 1 in. had occurred between adjacent courses. That corresponded roughly with the tolerances in the interlocking recesses. Up to that time it had been thought that the interlocking recesses were not entirely necessary to prevent sliding and that the frictional resistance between the concrete surfaces was sufficient; nevertheless those interlocking recesses had been included in the design of the Tranmere Block and also at the Victoria Dock, Hull.

From calculations which had been carried out after the survey it appeared that the angle of friction between the blocks must have been as low as $17\frac{1}{2}^{\circ}$ for sliding to occur. Normally an angle of friction of between 27° and 30° might be expected for moderately smooth surfaces of concrete. The reduction was in all probability due to the presence of silt mixed with oil which was continually deposited over the site and which no amount of cleaning would entirely remove. It had a fine and greasy nature which probably caused it to act as a lubricant between the courses. The survey also showed that there was little or no danger of any further horizontal movement since the tolerance in the interlocking recesses had been taken up. As a precaution, however, a further course of blockwork and a quantity of old rails had been added to the crest of the dam, in order to increase the frictional resistance.

In the places where relative movement had taken place between the adjacent blocks it was necessary to caulk the joints by diver. When that had been completed, the lock was successfully dewatered. Screened ashes were tipped over the face of the deck at intervals during dewatering, and for a period afterwards, to control the small leakage. As Mr Malcolm had mentioned, the leakage throughout the reconstruction work was quite small and had been dealt with by a single 6-in. pump, which Mr Dempsey believed worked intermittently.

Mr Cecil Peel (Assistant Chief Engineer, Civil, Port of London Authority) said that the Gallions Lower Entrance Lock had had rather a famous career. It had formed the major lock entrance of the Royal Albert and Royal Victoria Docks for many years, from 1886 to the opening of the King George V Entrance in 1921. It had carried a vast quantity of shipping, and it had still been used for shipping up to about 1939, although its condition had become steadily worse. Had it not been for the war, its reconstruction would probably have been undertaken at an earlier date.

The work described formed one of the larger items in the large programme of works which the Port of London Authority had carried out since the war, and it was considered that it would give a new lease of life to what was a very valuable entrance. In an impounded dock system it was obviously necessary to have an entrance large enough for the largest shipping likely to use the docks, but the existence of a smaller entrance was an invaluable asset because the majority of the ships were smaller in size than the largest. Gallions Lock had been found on a dimensional analysis to be capable of taking about two-thirds of the ships using the Royal Docks, and that was sufficient to demonstrate its importance.

Unfortunately, the old shape of the sills and invert had imposed a restriction on the depth available. The lowering of the sills and invert which had been carried out, although it had been costly, had given a valuable increase in that depth. The maximum size of ship which could use the lock would be about 510 ft long by 70 ft beam, with a maximum draft of the order of 29 ft 6 in. Formerly, such vessels could be locked at the Gallions Entrance only at high-water of spring tides, and at neap tides the maximum draft had been about 3 or 4 ft less. It would now be possible to lock vessels having a draft of 29 ft 6 in. at high water of almost every tide, which was a great improvement. A further consideration which would show the importance of that was the fact that each foot of draft of the vessel meant an additional 500 to 600 tons of cargo.

There were other important considerations in using the Gallions Entrance as a relief to the King George V Entrance. In the impounded system in question all the water had to be pumped from the river and, since the amount of water used at the Gallions Entrance was only about two-thirds of that used at the King George V Entrance, the more vessels that could be locked at the former the greater the saving in pumping the impounded water. Again, the water from the river was very silty and a great deal of that silt settled in the Albert Basin and had to be dredged. That meant that the water used for locking at the King George V Entrance was relatively clean, whereas that which would be used and which formerly had been used at the Gallions Entrance was relatively dirty, so that there would be a double saving, with less silt deposited and more dirty water run back to the river.

In choosing at the design stage the methods to be adopted to dry out the lock, the most important feature to be considered in connexion with the work had been the stability of the walls and floors. The design of the dams had been left to the contractors, but the installation of means to lower the ground-water level and to load the centre of the invert had been specified. That had been done after careful consideration and calculation, which showed that whilst the margin of stability of the walls was not large, it was sufficient provided that there was no free hydrostatic pressure acting on the back of them. Furthermore, since the invert had to be cut into it was necessary to lower the water pressure underneath the floor. It would have been possible, of course, to stabilize the walls against overturning or collapse by an elaborate system of strutting, but that would have cost a great deal of money and would have been a nuisance during the re-facing, so that the advantage lay from every point of view with lowering the ground-water level. Preliminary calculations showed that they could expect to get the water level down to something in the region of ± 80 works datum on the simple assumptions used. It was also considered that since the ballast in the bottom of the basin and on the bed of the river for a considerable distance from the lock was thickly covered with mud there was a good chance of obtaining an effective seal, and that the water could be taken down lower than the original calculations suggested. The actual results had been extremely satisfactory. The contractors had in fact put some struts across the lock as a precautionary measure, but during most of the time the lock was empty those struts had not been tight and had been carrying little if any pressure. The lowering of the ground-water level produced considerable settlement of the ground at the back of the walls, and no doubt that led to considerable cohesive drag on the back and so assisted in their stability.

It might be wondered why electrical machines had been installed to work the gates, instead of hydraulic. That was simply due to the fact that all the cranes at the Royal Docks were electric, and it was no longer necessary to maintain the many miles of hydraulic mains. The hydraulic system at the docks had therefore been remodelled and curtailed, and it had been felt opportune to install electric machines at the Gallions Entrance. Such machinery was commonly used on the Continent and in the United States, and he had no doubt that it would work equally well at the Gallions Entrance.

Professor A. W. Skempton (Professor of Soil Mechanics in the University of London, at Imperial College) remarked that there had been no reference in the Paper, or discussion, to the movements which took place in the outer cofferdam, and he thought they should be recorded. Together with Dr A. W. Bishop, Professor Skempton had been consulted by Mr R. Glossop, of John Mowlem & Co., concerning the design of the cofferdams for Gallions

Lock. Originally, both were intended to be block dams, but it was later decided that the outer dam should be a double sheet-pile wall cofferdam, largely because of the presence of the deep scour hole near the sill of the lock.

The outer cofferdam was designed on the basis of $\phi = 35^\circ$ in the filling and an angle of friction of 18° between that material and the steel-piling, and with a factor of safety of 1.4 against failure under an extreme head of 42 ft of water above the old sill. But even with that by no means excessive factor of safety, the section (see Fig. 2b, Plate 1) appeared to be perhaps a little too generous. Nevertheless, after the lock had been emptied, the crest of the cofferdam moved inwards as much as 14 in., under a maximum head of 37 ft of water, and so it was felt that the design was not over-conservative. The movement was progressively built up, reaching the value of 14 in. in March 1954, after which no further increase took place. A head of 37 ft or thereabouts occurred only at high spring tides and by the time that head was first developed across the dam, in November 1953, the crest had moved 12 in.

Analysis of the inner block dam showed the desirability of placing a slab of tremie concrete between the dam and the sill of the lock. That was also thought, by some, to be erring a little too much on the side of safety, but when the block dam shifted in the manner described by Mr Dempsey everyone had been very relieved that there was that strut, or rather slab acting as a strut, which would stop the whole dam from sliding.

It might be worth emphasizing the point that the friction between the concrete blocks had been so much less than expected. Had it not been for the interlocks there seemed to be little doubt that at least the upper portion of the dam would have failed, with the possibility of somewhat catastrophic events in the Royal Albert Dock basin.

* * Mr G. M. Cornfield (Civil Engineer, The British Steel Piling Co., Ltd) observed that on p. 143 there was a statement that the outer line of piling of the sheet-pile dam was driven about 15 ft into the chalk. From the other details given it appeared that the inner line of piling of that dam had penetrated the chalk to a depth of 25 ft. What was the size of the hammer used for driving those sheet-piles, and could any other relevant details be given? Was the driving easy or difficult? Could the Author give any more information regarding the relative costs of the inner dam (constructed of block work), and the outer dam (steel sheet-piling and filling)?

Mr T. F. Burns (Consulting Engineer) asked why blockwork was used for the inner dam? It could hardly be considered less costly than a double-skin steel sheet-piled dam. He would have thought that the casting of 8,500 cu. yd of concrete in blocks, the careful placing and subsequent removal by diver, and the relative slowness of the work would have made the engineers seriously consider the employment of a sheet-piled structure. The possible use of horizontal girders transmitting the load to the end of the lock walls or the use of diaphragms would both appear to be less costly than the blockwork construction.

Could the Author give approximate costs of the two dams, the time of their construction and removal, and had the concrete blocks any salvage value?

During the discussion Dr Skempton revealed that both dams had moved under full hydrostatic head. Could the Author supply any further detailed information on the extent of movement in relation to tidal and impounded pressure? That data might be very useful to those who had to design such structures. Presumably the deep wells in the outer cofferdam functioned throughout the work.

The period of reconstruction of more than 4 years seemed considerable, particularly in view of the importance of the lock from a revenue earning aspect. One slide was shown illustrating night work—was that general or had it only applied to certain operations and on certain occasions?

The employment of ground-water lowering methods was interesting, in fact it seemed

* * This and the following contribution were submitted in writing after the closure of the oral discussion.—SEC,

to be one of those jobs which would have been difficult to achieve successfully without the use of deep wells.

Apparently some settlement had occurred as a result of the deep-well pumping. Could the Author give some information as to the location and extent?

A rather cautious view seemed to have been taken by the Authority of the stability of the lock floor, since a margin of safety of 10-ft head from 82 to 72 was called for and, in addition, kentledge was placed on the floor. The kentledge apparently amounted to about 2.5 cwt/sq. ft of floor or the equivalent of about $4\frac{1}{2}$ -ft head of water. Did the Author consider that that latter provision was really necessary and if so would it not have been simpler to use ballast instead of supplying, placing, and removing kentledge, which he presumed was concrete blocks?

The Author had carefully set out the uses of the five grades of concrete but had given no corresponding mixes. Such information, with the crushing strengths specified, would be useful.

No particulars had been given as to the accuracy of the dressing of the quoin posts and meeting faces of the gates. There was often much difference of opinion as to the need for great accuracy in dressing of the greenheart; would the Author give his views on the subject and describe what was done in the present case?

Were any measures taken to overcome the effect of shrinkage of concrete skin on the walls and was the B.R.C. fabric placed near the exposed surface? To cast lengths of 140 ft in one pour seem to be excessive—had it proved entirely satisfactory?

Why were the Philbolts made as long as 3 ft? Presumably the function of those bolts was to secure effectively the facing to the body of the wall. Were any tests for adhesion made on the Philbolts and if so could the results be given?

The Author, in reply, said that a number of interesting points had been raised on matters which he had been unable to include in the Paper or had been prevented by restrictions of space from so doing. It would be realized that the Paper had been produced in September, while the work which it described had still been incomplete. However, the majority of the major construction work intended had, in fact, been done.

Mr Wild had asked whether, if the Author had to do the work again, he would do it in the same way. The Author felt that it was correct to say that the work which had been carried out at Gallions had proved very satisfactory and had shown that the original design, to which Mr Peel had referred, had proved to be perfectly sound. There had been a little concern until the dewatering had taken place, but after the preliminary excavations had been made the work had given no real cause for anxiety. To take away part of the floor of a lock, as had been done at the Gallions Entrance and to substitute another type of material, with uncertain shrinkage factors, had caused a good deal of anxiety until they had become accustomed to its behaviour; but he thought that, as all who had seen the finished work would agree, there was now almost a new lock, with a new wall face, new gates, and new machinery. That had been achieved at a cost which was only a proportion of what it would have cost to build a new lock in the same place under the existing conditions. Other things being equal, he would say, therefore, that if he had the job to do again he would make very little change from the way in which it had been carried out. A saving might be made on temporary works, but in general the major construction would be the same.

The deep-well system—had proved adequate—he would say even more than adequate, because although No. 7 well had never been used, and no pumps had been installed, the water plane in that area had been very little changed. If the work had to be repeated it should be possible to achieve a satisfactory result with a slightly wider spacing.

It was a little difficult to compare the conditions at Avonmouth with those at Gallions. The head of water was very much greater at Avonmouth than at Gallions and the velocities were very much higher. With regard to the discharge sluiceways of the outer apron, there were only two openings at Avonmouth as against six at Gallions. The Author had had, through the kindness of Mr Matheson, an opportunity to see the work carried out at

Bristol before undertaking the researches at Gallions. They had been fortunate in having that information to assist them. It was a little difficult to say that the methods of approach to the problem were quite the same, because at Gallions the amount of scour was not so serious as at Avonmouth, and they knew that on the removal of the outer dam they would be left with a very satisfactory rubble bottom as compared with the original chalk, which had proved to be soft. Also, owing to the lower water velocities, it had been thought, and he believed would be proved in practice later, that, if they could achieve a uniform flow over the apron by a minor modification of the sluiceways, they would have done all that was necessary to avoid future scour.

The matter really resolved itself into a question of economics. It would have been quite reasonable to assume, from the researches at Avonmouth, that all that was needed was to put in large amounts of rubble of good size at the scour-hole, or concrete blocks, or a pavement of some nature, and the water would not disturb it. If that had been done, however, it would have cost about £20,000. It would quite reasonably have prevented any further trouble, but what had in fact been done had been very much cheaper. The model had been built for about £1,700, including the cost of labour. The results found from the model were not, as Mr Allen had fairly said, to be considered a complete answer to researches on sluiceways, but they had given the information which was desired at Gallions, and putting that information into effect had cost approximately £3,000 to £4,000, giving a total cost of between £5,000 and £6,000 for all the work necessary to solve the problem. They had therefore saved about £15,000 compared with the cost of tipping in blocks, rubble, or large amounts of concrete, and that had been, from the point of view of an engineer, the most satisfactory way of dealing with the problem.

Mr Sivewright had asked for the rate of flow from the P.L.A. sluices. It could be taken as approximately 2,000 cusecs at maximum discharge. Measurements had been taken of the flow over the weir, illustrated in a slide shown by Mr Allen. The weir was situated at the far end of the outlet chamber. With regard to the altered time of locking by the construction of the sluices, they had not yet proved that point in practice, but he felt confident that by making a uniform flow through the sluices and sluiceways, and with the sluiceways all running full bore, the flow must be greater than would have occurred if sluiceways Nos 1 and 2 had been running full bore and the remainder only at partial capacity. He was of the opinion, therefore, that not only had they increased the output but the locking time was reduced by forming a truly even discharge and a satisfactory and uniform flow was achieved over the apron. By that means he thought that they had definitely added to the efficiency of the outer sluiceways.

With regard to the point raised by Mr Reed, they had had no results as yet on the operation of the machinery, because there had not been time to test it. The gates had been put into full use on Christmas Eve.

Mr Dempsey and Professor Skempton had referred to the blockwork dam. He agreed with Professor Skempton that they had been very pleased to have the horizontal interlocking sections in the blockwork. The friction between the blocks proved to be very low, as had been seen, and although he did not think that the Royal Albert Dock would have been the victim of a catastrophe, they had been glad to have something up their sleeve at that time. The original gates were still in position and would have proved effective. The water was being lowered and there was only about a 12 ft difference of head at the time that the dam started to move. There would have been time, he thought, to take all precautions to safeguard the Royal Albert Dock from being dewatered.

The Author thanked Mr Peel for his comments on the original work and also for having provided a very satisfactory design which the Author had had the pleasure and privilege to carry out.

The piling for the outer dam had consisted of Appleby Frodingham No. 3 section for the outer skin and No. 5 Larssen for the inner. A portion of the Larssen piles had been driven through the original scour-hole, and at the edge of that scour the penetration into chalk had reached 25 ft. Driving had not been exceptionally hard and had been carried out by a No. 9B₃ McKiernan-Terry hammer slung from a 15-ton derrick. The rate of driving had averaged between 5 and 6 piles per day. The piles had been 70 ft long and

the labour involved in handling and pitching accounted for much of time required for driving.

With regard to the relative costs of the two dams, the inner dam could be taken as costing about £550 per foot run, based on the effective length of the dam being 198 ft. The outer dam of steel sheet-piling and filling had a mean length of about 250 ft and the average cost per foot run was about 20% less than for the blockwork dam.

One of the principal reasons for the adoption of the blockwork dam at the inner position was the reduced space required, which involved less encroachment into the shipping area. In addition, that construction had not been under tidal conditions, and the provision of steel girders to transmit the load to the wing walls, as suggested by Mr Burns, would have had to be carried out entirely by divers.

The concrete blocks which had been recovered from the inner dam were being utilized by the Port of London Authority for the construction of another dam of similar dimensions at the Millwall Dock.

The deep wells provided in the dams had not been utilized during the major part of the construction.

Night work had not been customary and the sodium lighting illustrated had been provided by the Contractor for normal winter conditions.

Some settlements had occurred as the result of deep-well pumping and the reduction of water level behind the lock walls. The settlements had been limited to the general ground level immediately behind the lock walls, and had reached 9 in. in places.

The placing of the kentledge on the floor of the lock had been decided upon as a safeguard and had had to be undertaken before the result of the deep well pumping could be proved. The lowering of the water level by those deep-well pumps had been more effective than had been anticipated and, in consequence, some of the kentledge had been additional to requirements.

Concrete blocks had been used in preference to ballast in view of the large amount of work necessary on the floor of the lock for the construction of the knees, which would have required the ballast to be closely confined.

Table I showed the concrete mixes and crushing strength specified.

TABLE I

Grade	Minimum resistance to crushing: lb/sq.in.		Water/cement ratio	Volume of aggregate to 112-lb. cement, allowing for 25% bulking			Slump range
	7 days	28 days					
				Cu. ft All in	Cu. ft Coarse	Cu. ft Fine	
"A"	—	—	—	12½	—	—	1-2
"B"	900	1,500	0.70	10½	—	—	2-3
"C"	1,700	2,500	0.60	8½	—	—	2-4
"Cg"	1,700	2,500	0.60	—	5	2½	3-4
"D"	2,000	3,000	0.50	—	4	2½	4-7
"Dg"	2,000	3,000	0.50	—	3½	2	4-7
"E"	2,500	3,750	0.45	—	3½	2	4-7
"Eg"	2,500	3,750	0.45	—	3½	1½	4-7

The dressing of the concrete had been carried out to an accuracy of about 0.006 in. The granite work had been completely re-built and the conditions had been suitable for the best possible workmanship. Excellent conditions of gate tightness had been achieved and the Author considered the high degree of accuracy specified to have been justified.

No special measures had been taken to provide for expansion or shrinkage on the face of

the lock, for the majority of the work was under water, and the variations in temperature were limited.

The Philbolts had been made as long as 3 ft because of the poor condition of much of the concrete in the original walls. Tests had been carried out on three typical Philbolts which failed under test at 6·2, 4·2, and 8·2 tons respectively.

The closing date for Correspondence on the foregoing Paper has now passed and no contribution other than those already received at the Institution will be published.—Sec.
