Costs.

The estimated cost of the widening work was £400,000. During the contract it was decided to close the south end of No. 9 dock and form a fitting-out berth along the north wall of No. 4 tidal basin, at an estimated cost of £40,000. This work was added to the main dock widening contract and the final settlement for this was just over £400,000. Departmental work on fendering, railways, roads, water mains, etc., amounted to £30,000.

Acknowledgements.

Messrs. Nuttall were represented on site throughout by Mr. J. A. L. Hooper, B.Sc., and the Author wishes to acknowledge the very pleasant spirit in which the work was carried out It could so very easily have been quite the opposite, because the site was in the heart of an important dockyard, the activities of which had to be maintained. Interruptions by ship movements in the basins at each end of the dock were inevitable; at one time two aircraft carriers were docked in the adjacent dry docks less than 200 feet away; dockyard traffic—trains, lorries, cranes, and personnel—crossed the contractors; and the main pumping system was common to all dry docks—No. 10 included. To offset this, however, facilities existed in the dockyard for carrying out major plant repairs of almost any kind and, as the Contractors were always reasonable in their dealings with all Admiralty departments, these facilities were readily made available to them. They were not cheap, because Admiralty charges to "private individuals" are purposely high so as not to encourage them, but at times their promptness was invaluable.

In short, the Contractors became almost another department in the Dockyard and entered into its "give and take" life, to the mutual benefit of everyone. Apart from a delay in the completion of the caisson by another contractor, the work was completed on time; no arguments over interruptions arose; and at the final settlement the value of disputed items was less than £2,000.

The Paper is accompanied by two sheets of drawings, from which the Figures in the text have been prepared.

Discussion.

The Author introduced the Paper with the aid of a series of lantern slides.

Mr. D. P. Bertlin observed that the arch dam at the head of the dock, which was a temporary dam for carrying out the work inside, was a form of construction which had not been employed very often. There were two main advantages in constructing it in a semi-circular form, namely, that the reactions at the dock heads were normal, so that no special keying was required to take any side thrust, and that, as the reaction at any section was along the axis of the dam, there was no possibility of any tension being set up, so that a very slim section could be adopted. When Messrs. Nuttall put forward the scheme of a 5-foot wide dam, it was with those facts in mind, and Mr. Bertlin considered that that design was perfectly sound. On the other hand, he would not dispute the Admiralty's requirement of an extra 5 feet of concrete, as a large factor of safety was important. Moreover, the relevance of the slenderness-ratio arose. The analogy with an arch dam design broke down because normally, in a permanent arch dam, the width at the bottom was much less than at the top, so that the wide portion of the dam was over a short span and the question of slenderness-ratio did not arise.

The sheet-piles were driven to within 2 inches ; bearing in mind that they were driven in the middle of the dock on a curve, that said a great deal for the foremen concerned. The importance of making the dam truly circular, so that no tension arose, would be appreciated.

With regard to placing the concrete, leaving holes in the dam for the explosives proved to be a rather difficult problem, because the plan of raising the pipes as the concrete was formed did not prove satisfactory. If the pipes were drawn too quickly the holes became filled with concrete, whilst if they were not drawn quickly enough they stuck and became concreted in. The question of blowing the dam after the work was finished had been dealt with by Mr. James Lorimer,[1] who had suggested that the pipes should remain in. One difficulty in that connexion was that fragments might be left over after the blowing, which might prove difficult from the dredging point of view ; moreover, the pipes would be in the way during the concreting process. Mr. Bertlin considered that the best plan was to leave them out altogether, to concrete with large skips, and subsequently to drill holes for the blowing. The drilling was rather difficult, so that even that was not an ideal solution, but he believed that it was important to avoid interference with the concreting operation.

He believed that the deposition of under-water concrete in quantity had been first practised in Germany, and had also been used occasionally in Great Britain ; for example, in the construction of the Dover train-ferry dock. A number of problems, however, remained to be solved. Mr. Bertlin's experience was that the method of using a skip with a skirt had proved to be the best.

He wished to endorse the view of facing concrete expressed by the Author, who had mentioned that the results were quite as satisfactory if the idea of having a foot or so of facing concrete in front of the wall was not adopted. Mr. Bertlin believed that contractors would agree that,

[1] " The Use of Explosives in Civil Engineering," Instn Civ. Engrs, Works Construction Paper No. 3 (1946).

from the constructional point of view, facing concrete was far better left out.

On behalf of Messrs. Nuttall, he wished to thank the Author for the kind remarks he had made in his Acknowledgements. It was due to the attitude of the Author and of the Admiralty Departments towards the whole of the work that Messrs. Nuttall had been able to carry it out in the spirit which had been mentioned in the Paper.

Mr. Cecil Peel observed that venting the floor instead of strengthening it to withstand hydrostatic pressure was a common practice in connexion with dry docks founded on rock; but at the Royal Docks of the Port of London Authority, two dry docks which were founded upon ballast, as opposed to rock, were vented. In 1914, the Western dry dock had been widened from 56 feet 6 inches to 80 feet and lengthened from 408 feet to 575 feet. He believed that the work had been carried out in a manner similar to that adopted at Devonport, by building a new wall on one side at the back of the old wall and widening the floor. The original floor, which was rather thin, was designed to sustain hydrostatic pressure, but the widened floor was vented with two rows of 4-inch pipes, 50 feet apart. Those pipes were fitted at the top with loose cap plugs, which were free to lift and allow the escape of water, but, needless to say, they had become completely choked up; they were quite dry and no water did, in fact, escape. He had had one opened up recently and had found that the water merely stood in the hole at floor-level, so that either the floor was continuing to behave as an arch or the pressure never really developed, possibly owing to leakage through cracks in the walls. As the King George V dry dock, on the other hand, although the floor was made thick to withstand hydrostatic pressure, vents were provided, 50 feet apart, through the side walls, and in that case water certainly did come in; in fact, the 6-inch pipes ran at about half-bore. Flaps were provided, but, as was usually the case, the flaps did not act as such. The floor of the King George V dry dock was 39 feet below the constant water-level in the neighbouring wet dock, and the water-level in the ground round the dock stood at about 15 feet above floor-level when the dock was empty and rose, when the dock was flooded, about 10 feet after a day of flooding. The ingress of water through the vent pipes was known to have brought in a certain quantity of fine silt, doubtless from the surrounding ballast. That emphasized the fact that, if venting were employed, at all events in alluvium, a proper and adequate system of filtration should be employed, as otherwise there would be a risk of loss of soil and consequent settlement. Therefore, if a dock was built in alluvium and was to be vented, it seemed best to vent it through the walls and not through the floor, as it would be very disadvantageous to lose soil from under the floor.

The section of the new east wall at Devonport, shown in *Fig. 3*, appeared to be extremely heavy, particularly near the top of the wall. Would not it have been more economical to use a section more like that shown in

Fig. 4 or *Fig. 5* and to build brackets or counterforts within the trench to carry a longitudinal crane beam ?

Could the Author give some indication of the size (height, length, and width) of the sections of concrete that were deposited at one operation, and state whether any particular methods were adopted at the joints ? Were any expansion joints provided in the walls, and had contraction caused the construction joints to open up ? That kind of thing usually happened and water entered, and it provided a very valuable form of vent. Was the surface of the set concrete keyed in any way ? Was it skinned or chipped ? Mr. Peel had always considered that the best type of bond could be obtained by completely skinning off the surface and soaking the concrete, and then brushing cement grout into it before any new concrete was put in.

With regard to the concreting of the dam under water, the tube mentioned by the Author appeared to have been used as a skip, but would not it have been better to use it as a tremie pipe, keeping the mouth of the pipe continually buried in the concrete and feeding down from the top ?

Mr. G. A. Wilson observed, with regard to the caisson stops, that the width of joint between the granite stones had not been given in the Paper ; in the Singapore graving dock $\frac{3}{8}$ inch was normal and $\frac{1}{4}$ inch was used between the granite stones forming the caisson stop. The tolerance for the surface of the caisson stop was described in the Singapore specification as " the thickness of one sheet of thin foreign notepaper " ; actually 0·003 inch was used, and it was interesting to note that at the Devonport dock a 30-per cent. increase in tolerance was allowed, 0·004 inch being specified.

The production of a batch of concrete every $3\frac{1}{2}$ minutes by one mixer working day and night for six days, was remarkable. The figures given in the Paper showed that, at the rate of 100 skips per shift, 2,400 cubic yards of concrete was deposited ; but, from *Fig. 7*, the quantity up to the construction joint just above low water, appeared to be 3,600 cubic yards, so that not only was the job excellently organized, but it was almost magical in that particular respect.

The most controversial aspect of the Paper was, perhaps, the thickness of the wall. The ground seemed to have been good ; in one case 35 feet of rock had been taken out, and yet a heavier section of wall had been used than the one that had stood for 30 years. At one period it was advocated in the Civil Engineer-in-Chief's Department of the Admiralty that the top of a dock wall could be widened to carry the crane track, and the bottom of the wall reduced in thickness if the dock were designed as an inverted arch. When it came to construction, the narrow base to the wall was usually ruled out, because in bad ground it certainly was difficult to build ; but it seemed possible that something on those lines might have been done or, alternatively, that the crane track might have been carried on a row of piles and a beam, as had been done on the west side of the dock at Devonport.

The Author had stated that the heavy section had been adopted to prevent settlement of surrounding buildings, but from the slides there appeared to be no buildings of any importance in the vicinity. In any case, the prevention of settlement seemed to have been dealt with in an expensive way.

The concrete mix ($5:2\frac{1}{2}:1$), seemed unnecessarily rich. At both Singapore and Portsmouth Mr. Wilson had built docks with an $8:1$ concrete, the specification providing for 8 parts of coarse aggregate to sufficient sand to fill the voids plus 12 per cent. and 1 part of cement, which approximated to a true $8:4:1$ concrete, as opposed to the enriched $6:1$ concrete which the Author had used. Taking a present-day price of 39*s*. 10*d*. for the $5:2\frac{1}{2}:1$ concrete and 36*s*. 3*d*. for the $8:4:1$ concrete, the use of the latter would represent a saving of about £15,000. In addition to the use of that rich concrete for the wall, 6 inches of facing concrete, with proportions of $2\frac{1}{2}:1\frac{1}{2}:1$, was specified. The object of the facing to a dock wall was to provide water-tightness, wearing surface, and appearance, and Mr. Wilson considered that a $5:2\frac{1}{2}:1$ concrete should be quite good enough to give a face which would stand up to those requirements. If a facing concrete had to be used (in his experience it was a serious inconvenience in construction), he believed that a weaker concrete could be used in the main mass of the wall. In an Admiralty dock built recently at Colombo, the wall had been faced with blocks of about 2 feet cube, pre-cast, in $1:2:4$ concrete. Those blocks had saved front shutters, and a very satisfactory finish had been obtained.

Mr. Harry Ridehalgh observed that forcing up of the granite paving from the main body of the concrete was apparently a fairly commo occurrence. During a recent discussion with the Author, it had been suggested that the water percolated gradually through the lower thicknesses of the concrete and built up a pressure-head under the denser granite or concrete paving, finally bursting it off. That difficulty might be overcome by abolishing the use of those pavings and building the whole thickness of the floor in concrete of a uniform quality.

It seemed unusual that the width of a crane-track should determine the width of a dock wall at coping-level. Apparently the crane-track on the other side was successfully carried on piles, and he considered that similar means might have been adopted for the new wall.

The Author had stated that the profile of the wall had been determined by shipping requirements, and that the cantilever altar was one of its features. In the construction of the dock at Colombo to which Mr. Wilson had referred, similar cantilever altars were used, and it was difficult to devise a satisfactory shutter. Eventually a rather complicated arrangement had to be provided, after a suggestion that pre-cast altars should be used had been abandoned owing to handling difficulties.

Mr. Ridehalgh had always considered that the degree of accuracy with which it was attempted to dress dock sills was unnecessarily high, particularly in view of the fact that sooner or later inequalities would arise in the surface of the greenheart and debris would be trapped from time to time between the granite and the greenheart. He knew of one or two docks which had been built with a greenheart face abutting direct on in-situ concrete, and in several cases recently rubber inserts had been provided in the caisson face abutting against pre-cast units; they were perfectly satisfactory, so far as he was aware, and they would certainly be less trouble during the construction period.

Had any successful means been adopted for draining the crane-track and preventing corrosion? What was the reason for abandoning the sliding caisson in favour of the ship-type caisson? And did the cost given for the whole work include the cost of the caissons?

Mr. J. H. Jellett inquired whether, when the design was being prepared, it had been anticipated that any particular quantity of water would flow from the vents. In his experience the engineer-managers—who were the functionaries who ran the pumping stations in Admiralty dockyards—always looked askance at the entry of water into docks on a scale which might involve them in running the drainage pumps for any considerable period of time.

The Author had stated that the functioning of the non-return valves, after six years of existence, appeared to be rather doubtful. In view of that statement, were they being retained in the dock and put into condition again, or had it been decided to abolish them altogether?

In view of the increased span across the entrance, it was clear that the horizontal arch carrying the sill must have required a certain amount of redesigning, and it would be interesting to know how that had been done, whether any increased depth of arch had been found to be necessary, and how far it had been necessary to cut into the old wall to secure an abutment for the new arch ring. A cross-section through the whole dock to illustrate the position after the reconstruction had been completed would be illuminating as showing how the new width specified had been achieved, because, although it was stated in the Paper that the dock was to be widened in the barrel from 121 feet to 151 feet, the distance between the new and the old coping-lines shown in *Fig. 3* scaled only 22 feet 6 inches, instead of 30 feet. Could the Author give some indication of how the full new width had been achieved in the final dock.

The widening of the dock must have increased very considerably the quantity of water requiring to be pumped for every docking, and, as the dock was being operated by the old pumping-station, presumably some increase in the length of the docking time had had to be accepted, unless new pumps to deal with the larger dock had been fitted.

Mr. T. L. Coffin observed that, in view of the fact that about 7 years had elapsed since the completion of the dock described in the Paper, it would be interesting to know how various features of the design had stood up during that period. On the Author's suggestion, facing concrete had been omitted in many places, and it might be possible for him to state whether or not its omission had been justified by the course of events during the last few years.

With regard to the demolition of the wall of the dam, the Author had stated that he did not consider that any real trouble had arisen from vibration or shock from the explosives. Had that been borne out during the course of the past few years ?

Mr. Coffin had been helped considerably by the excellent record drawings made by the Admiralty during the whole course of the work. which had enabled the trouble which arose on measurement through scrappy record drawings to be avoided.

Mr. C. F. Armstrong observed that apparently trouble had been experienced in constructing the concrete arch satisfactorily, and further trouble in removing it. Did the Author consider that the cost had been justified ? Most of the competing contractors had offered to construct the standard cofferdam with two rows of sheet-piling filled with earth, and he presumed they had good reasons for doing so. Had the use of the concrete arch been found to be an economical idea, and would the Author recommend its use on another occasion ?

In *Figs. 4* and *5* a salt-water magazine-flooding main was shown. Could the Author explain the function of that main, and state why one had not been fitted in the wall of No. 10 dock—or, at least, had not been indicated in the drawings ?

In Item No. 3 of the " Order of Procedure " (p. 10, *ante*), it was stated that the Admiralty was to drain the space between the south end wall and the caisson, and that the contractor was to drain the space between the dam and the caisson : presumably there was a good reason for that provision, and perhaps the Author would explain it.

Mr. R. W. A. Fane observed that he had been particularly interested in the Author's account of the construction and subsequent destruction of the cofferdam (pp. 14 and 23, *ante*). In 1937, the Admiralty had let two dock widening contracts, namely, that at Devonport to Messrs. Nuttall and the other, for a similar job at Gibraltar, to John Cochrane and Sons, Ltd. On the latter contract Mr. Fane had been made responsible for the construction of the temporary cofferdam, and he wished to make certain comparisons and ask two questions.

The Devonport cofferdam appeared to have been founded on a rock bottom, whereas the Gibraltar cofferdam had been founded on a bed of silt about 15 feet deep, and it was largely for that reason that at Gibraltar a cofferdam of what had been described as the " conventional " type had been used, with two parallel rows of Appleby-Frodingham III piles, 34 feet apart, and three tiers of double 8-inch by 6-inch rolled steel joist walings. The walings were second-hand, having previously been used for Waterloo bridge. Normal tie-bars had been used, $2\frac{1}{2}$ inches and 3 inches in diameter, and the dam had then been filled with earth from excavations behind the dock wall. The original plan had been to fill the dam with rock and quarry debris, but eventually—largely with the object of saving time—the contractors had put in everything that they could find. All the walings and tie-bars had had to be put in by divers, and, although seven or eight divers were at work and considerable trouble was taken to make simple connexions for the tie-bars, the construction of the cofferdam was very slow and expensive. However, once built, that traditional cofferdam had answered the purpose extremely well, with surprisingly little leakage. When the dock was first pumped out and the cofferdam took the full load of approximately 40 feet of water pressure (at Gibraltar the tidal range was only 3 feet, whereas at Devonport he believed it was more than 20 feet), a very slight inward movement developed and the centre of the dam came in about 6 inches during the first three months; but it then settled down and no further movement occurred. A number of holes were cut in the inner row of sheet-piles, to allow the muck filling to drain. Probably that drainage caused the movement, because the cofferdam was absolutely stable after the first three months.

It was obvious from the Paper that Messrs. Nuttall had had a very trying time with the blasting to remove their concrete cofferdam, whilst at Gibraltar no trouble had been experienced with the removal of the more conventional type of cofferdam.

Was it fair to say that the expensive decision to strengthen the concrete cofferdam with an inner ring of concrete 5 feet thick over the bottom 25 feet in height was entirely due to the first sector of the main concrete dam being found to be faulty? Mr. Fane believed that it was generally accepted that extremely good concrete could be cast under water provided that it was cast in absolutely still water. Possibly the rise and fall of the tide had caused a difference between the water-level inside the inner row of piles and that outside the outer row, resulting in some flow through the clutches of the piles. Presumably that accounted for the loss of cement and for the non-homogeneous concrete of which that sector of the dam was found to be made. What method had been adopted for keeping the water-level inside and outside the piles the same? Possibly some 12-inch diameter steel equalizer pipes put in by divers near the bottom through both rows of sheet-piles, with a flange on the inner side for sealing up, would have kept the water-level more nearly the same inside and outside the dam, and so would have reduced the flow of water and improved the quality of the concrete.

For demolition blasting it was generally accepted that a homogeneous material was essential, and he wondered whether the equalizer pipes or something like them would not have made it easier for the contractors to remove their dam; firstly, by giving a concrete of better quality, and secondly, by giving additional holes for the insertion of the blasting charges.

From bitter experience he could endorse Mr. Bertlin's remarks on the extreme nuisance value of a little facing concrete up the front to the contractor who had to do the mass concreting.

Mr. J. A. L. Hooper observed that the work described in the Paper had been carried out in the heart of a busy dockyard, where space was very limited, and apart from the ordinary troubles of the job, that had entailed arrangements to fit in the traffic of materials—excavation going away and concrete materials coming in—with the ordinary work of the dockyard. Moreover, the work had been done at a time of re-armament, when the dockyard was very busy and contained many ships, some in full commission, with many men going to and fro all the time. That had made it very difficult to keep the work going at times, but the difficulties had been overcome.

The blasting of the existing wall in so confined a space was also a problem, as much of the material was granite, which flew about considerably, and it could not have been done successfully without the use of torpedo-nets to catch flying fragments.

There was a considerable amount of granite work at the entrances, as the two grooves necessitated the dressing of four watertight faces. Moreover, the old groove sills had to be removed from the bottom and up the sides of the west wall, and the new stones had to be inserted before the dressing was done. He had been rather apprehensive that, owing to the blasting, the joints of the existing concrete might become open and that, during testing, water might find its way round the back of the stones, or even lift the sill; but actually no leakage appeared round the backs of the stones, and the sill did not lift at all, although the entrance was very wide. That was due to the excellence of the existing work, which had been done forty or fifty years ago, when the dock was built, and to the care taken in the cleaning and the mortaring around the new stones when they were set.

The contractors had experienced trouble with the dam at the beginning, but they had had confidence in it and had felt safe in working behind it—a very important consideration when a large number of men were working in a deep trench.

There was a wide difference between the Larssen piles used on the outer side and the Universal piles used on the inside. The latter were used on the inside because they would be wanted for trench walings later; but, curiously enough, the cement found its way through the locks of the Universal piles and not through the locks of the Larssen piling.

He believed that, given the same conditions, the contractors would use the same type of dam again, although admittedly it had cost more than had been expected.

In conclusion, he wished to say that, owing to the difficulties of the work, the contractors would not have been able to finish it in time without the help of the Author and of the other dockyard authorities.

*** **Mr. J. Collett Dickinson** observed that rich facings to structures so frequently used by designers, might :—

(*a*) in some cases, such as the water face of gravity hydraulic structures, serve a specific design purpose ;
(*b*) in some cases, such as surfaces subject to the action of high-velocity water, offer economy in material ;
(*c*) merely represent a waste of labour and material, accompanied by high nuisance value in actual construction for a problematical return in appearance ; or
(*d*) in many cases represent bad structural design.

In the interests of economy, if for no other reason, designers should examine very closely the purpose served by rich concrete facings in any particular design. Mr. Dickinson suggested that such facings on the inner face of a gravity dock wall actually came under paragraph (*d*) above.

The concrete mix used appeared to be unduly rich. If density were the object, it was often not appreciated how much could be achieved by the use of an admixture of "crusher dust" or "rock flour". That method had been employed by Mr. Dickinson in reinforced-concrete underground reservoirs, with excellent results. The loss of strength was small and certainly would not reduce the strength of an 8 : 4 : 1 concrete below that necessary in a massive gravity wall.

It was interesting to note that the quoin stones were set on wooden wedges. That was the practical method ; but would the Author state whether the specification had permitted the use of wooden wedges for that purpose ?

Mr. J. M. P. Hooley observed that he presumed that in the Table (p. 5) giving water-levels in the trial pits, "D (East)" should read "B (East)".

The difference in the water-levels existing in the two trial pits mentioned was interesting. It might be assumed that most of the rainwater falling on the area around No. 10 dock was led away by drains, and the difference in water-level in the ground on the two sides had to be accounted for by a difference in leakage occurring from the basins or from underground springs.

Six months after the first readings were taken it appeared that the water-level on the west side had fallen 1½ foot, whilst that on the east had risen 4½ feet. That would seem to need some explanation in view of the general watertightness of excavation on the east side of the dock.

No mention had been made of any pumping, but Mr. Hooley assumed that a considerable amount was necessary initially to reduce the standing level of the water from 21 feet below cope to the lowest depth of excavation.

*** This and the following contributions were submitted in writing.—SEC. I.C.E.

In what form had "the three small springs" occurred—was the shillet fissured at that point, or was there any change in the formation there? What remedial measures were taken, and was it necessary to plug or drain away the springs? It would seem that an opportunity was afforded to use the pressure grouting which had proved superfluous to the point of refusal when attempted elsewhere.

In describing the attempts made to ascertain the nature of the concrete in the dam from which so much cement had been lost, the Author had stated that a trench 12 feet long by 3 feet wide was dug to a depth of 20 feet. At that depth the really bad concrete had not been reached, as most of the laitance found outside the intrados piling had, presumably, been formed in depositing the concrete on the lower half of the dam when the 18-inch-diameter tube was in use.

The Author's statement (p. 15) that a slight "negative pressure" applied to the dam owing to the water-level inside the dock being 1 foot higher than outside, was sufficient to open up all the construction joints, would seem to suggest a simpler means of dismantling such structures in future. In that instance it might have proved of considerable assistance to loosen up the construction joints by such a reversal of pressure before blasting was resorted to.

Mr. Hooley appreciated that the use of a concrete arch dam as a temporary structure was experimental and involved certain novel features, but he would be interested to know whether its adoption really was, or could be made, the means of saving money and/or time in comparison with the orthodox type of gravity cofferdam.

He also wished to inquire as to the nature and extent of the damage to permanent work occasioned by the use of the 15-lb. plaster charges used in the removal of the dam.

Since the joint between the sill stones and the supporting concrete was frequently the part of a dock most vulnerable to water seepage, the difficulty of bedding the prepared granite blocks should receive further attention. Mr. Hooley presumed that reliance was placed upon the 2½-inch diameter bolts to prevent the sill stones from lifting.

Considerable trouble and expense had been involved in the re-use of granite coping blocks. Whilst that could be excused partly by a desire to make the new wall match the one on the other side, it should be regarded as an extravagance, as there was little reason for the coping to be seriously damaged, and even if it were, it would be easily repaired.

Mr. Hooley presumed that, for the resistance moments of the old and new sections of the dock wall, values should have been given in "foot-tons per foot" not "tons per foot."

No mention had been made by the Author of any attempt to ensure that the wall and floor would act as one, and if it were assumed that no shear resistance was provided at the junction of the new and existing work, the stress on the underside of the toe of the wall would be high. If a stress diagram were to be drawn for the pressure on the rock formation under the wall and floor, with the dock in the condition of being empty and with a vessel docked (giving, say, a superimposed load of 32 tons per lineal foot on the centre-line of the floor), it would show a sudden change from approximately 7 tons per square foot at the underside of the toe of the wall to approximately 1 ton per square foot at the underside of the edge of the floor. Such a change in stress might be permissible where, as in that case, the dock was founded on rock, but it would be very undesirable where a softer material was experienced. Was any account taken of that in preparing the design ?

The section of the wall was unusually heavy, for the reasons given by the Author. With the section adopted the weight per lineal foot had been increased from 92 tons to 126 tons, representing a total addition of approximately 17,500 cubic yards of concrete in the wall of the dock.

The alternative of cantilevering the outer crane rail from the back of the wall, as shown in *Fig. 5*, appeared attractive, and should result in a considerable saving, despite the fact that a greater section of the trench backfill would remain for consolidation after the withdrawal of timbering. By keeping the withdrawal of timber and the consolidation of backfill in step with the advance in the wall concreting, it would be possible to cast the cantilever section supported on that filling and without added expense in shuttering. Moreover, the projection of that cantilever behind the wall would serve to mask effectively the results of any slight settlement in the filling which might occur subsequently.

Mr. R. Graham Keevill observed that the usual methods had been applied for the design of the new dock wall. The wall was assumed to be an independent unit, no support being given by the floor. The ground behind the existing walls was known to be shillet with water originally standing in it at a level of a few feet below coping-level. It was not known whether water would flow freely through the shillet from some unknown source and necessitate heavy pumping during the progress of the work of reconstruction. Pits were therefore sunk to obtain, if possible, some knowledge of the probable quantity of water to be dealt with. During the sinking of the pits large quantities of water were pumped out, with the ultimate result that the ground around No. 10 dock was drained. It was anticipated that in future years the strata would again become waterlogged, and allowance was made for that eventuality in the design of the new wall.

The Author had referred to an old scheme for widening by cutting away the altars of both walls and strengthening the walls by adding new concrete to the backs of the walls. The difficulty of bonding old and new concrete was well known. After careful investigation of the vertical and horizontal shear stresses within the wall, it was apparent that both side walls would probably have to be taken down to nearly floor-level and rebuilt.

In 1936, the widening of No. 10 dock at Devonport and of No. 1 dock at Gibraltar became urgent, and no time was available for further argument. The Admiralty departments concerned had stated their requirements and Sir Athol Anderson, K.C.B., M.I.C.E., then Civil Engineer-in-Chief of the Navy, approved the proposal for widening both docks by building new walls behind the old walls on one side of each dock, and widening the entrances on the same side as the new walls. The contract drawings for both docks were completed by a small temporary staff at the Admiralty in about nine months.

In 1923, when the Singapore naval base was in the preliminary stages of design, the Admiralty departments concerned concurred in an entirely new arrangement of altars for the 1,100 foot dry dock which was built there in subsequent years. That dock was intended to accommodate modern naval and merchant ships of the largest size and had, in fact, docked the Cunarder *Queen Mary*. That departure from the old type of naval dock decided the general profile of the new walls for the reconstructed docks at Devonport and Gibraltar, and all later proposals for other docks. The docks looked strange with an old-fashioned wall on one side and a modern wall on the other side, but they were efficient, and the cost was considerably lower than that of new docks.

When the width of the widened entrance at No. 10 dock was being considered, the final decision was based on the fact that the tidal entrance from the Hamoaze to the closed basin was 125 feet, and that decided the greatest width of any ship that could be docked at Devonport. The width and profile of the widened entrances to No. 10 dock was made the same, and the floating caissons used in both entrances were interchangeable. The entrance width and profile of the new wall decided the width of the reconstructed dock at coping and floor-levels, because the face of the altars of the new wall were kept on or just behind the line of the 1-in-12 batter of the new entrance wall.

The Author had compared the weights of the old and new walls ; but they were not really comparable, because the old wall did not carry 50-ton cranes with their wheel loads of 200 tons on a length of 12-feet, alternating either on the back or front of the wall. Nor did the old wall give the accommodation afforded by the new wall.

Built in the new wall was a subway for pipes and electric cables, stairways from floor to coping, with tunnels from its landings to the altars, a magazine-flooding culvert with outlets to hydrants, and a drainage culvert. The design also provided for chambers in the upper part which could have been used for stores, air-raid shelters, etc. ; but apparently the wall had been built without them. It would be more correct to say that the back of the wall was modified to suit the contractor's proposals for timbering the trench. When the new wall was designed little information was available in regard to the effect on a wall of a bomb bursting on or near it, and no reasonable allowance could be made in the calculations for such an event; but it was borne in mind that the wall should be as efficient as possible and remain in position when possibly a naval vessel costing many millions of pounds would be in the dock for urgent repairs.

Docks of the future might have to be built with much stronger walls, and hydraulic and earth pressures might not be the only deciding factors.

Mr. E. G. Walker observed that one of the most remarkable features of the work was the composition of the concrete. It was difficult to understand how a fine aggregate containing 10 per cent. of china clay flour, with mica in addition, could be permitted on even second-rate work. The earlier experience of Admiralty standards of civil engineering construction was that they were much higher than those which commercial undertakings and other dock owners could afford to adopt. Yet, from the Author's statement, material which should be rejected for decent cottage building was used in work of primary importance. Great Britain was favoured by the availability of good concrete aggregates, and had not the reason for using poor material that was valid in some countries, namely, that suitable material could not be obtained within a practicable distance of the working site. Therefore, the use of bad material should be condemned all the more strongly.

It was probable that much of the excessive laitance, of which the Author complained, was cement and clay and not simply fine cement as he had appeared to assume on p. 14. Possibly the unsatisfactory finish of the concrete surfaces described on p. 16 was a result from the same cause.

The Author, in reply, observed that, as different speakers had covered similar points, he proposed to reply under the heading of various items of the work rather than to individual contributors.

Wall.—A salt water magazine flooding main was included at Devonport and should have been shown on *Fig. 3.* When warships were at sea magazines could be readily flooded by opening the appropriate sea cocks. Normally, ships were "de-ammunitioned" before dry docking, but should that not be possible the magazines could still be flooded, if necessary, from the magazine flooding main, the sea cocks being connected with flexible hoses to branch pipes off the main.

The mass concrete mix used at Devonport was equivalent to 7 parts of mixed aggregate to 1 cwt. of cement, compared with 12 : 1 at Singapore. At Devonport, the concrete was quite rich enough, but the Author understood that at Singapore it was too poor to hold back water adequately and for future work he would propose a mix of 8 : 1 (8 parts mixed aggregate to 1 cwt. cement) without any special facing concrete. It had been feared, at Devonport, that a surface formed with a $2\frac{1}{2}$-inch aggregate might not wear satisfactorily, but a short section of the wall was built as a trial without special facing concrete and six years later it was impossible to distinguish it from the rest.

Concrete was placed in lifts of about 4 feet and in four timbering bags at a time (namely, in lengths of 52 feet), except the bottom lifts, which were specified to be placed in one thickness of 12 feet; there the lengths were 26 feet. Each horizontal joint was stepped 12 inches down to the back to form, theoretically, a shear key, but actually the step was more important as a water bar. Vertical joints were stepped back 2 feet at every other lift, except at the two-third points along the length of the wall, where the joints were in one vertical line. They were chased on plan to form a shear key and a continuous column of bitumen was incorporated as a water seal. They were very successful and it would have been an improvement if all vertical joints had been like them. Altar positions were decided by the Naval Architects and were cantilevered to keep the net offsetting of the face of the wall down to the minimum.

It was not fair to compare the weight of the new wall (126 tons) with that of the old one (92 tons), because the permitted shape of the latter was much more favourable to stability than the almost vertical front of the former. Proper considerations were: (*a*) was the wall the correct type for the site? (*b*) if so, was it a good example of its type? The correct type for a wall was governed by site conditions. *Fig. 4* showed an Admiralty design for a stock proposed (but not built) at Belfast, where there was 40 feet of "slutch" (very soft, wet clay) overlaying stiff clay. It was intended to drive steel sheet-piling to the stiff clay and to take the latter out under normal trench timbering methods. The vertical back of the wall would have been back-shuttered off the sheet-piling. Additional piling for the crane and rail tracks well behind the wall was necessary to keep surcharge loading (through the "slutch") off the wall. Taking concrete at 140 lb. per foot, a wall 17 feet deep to that design would weigh 100 tons per foot. Separate piling for the crane track would have cost £20 per foot, for which sum another 10 tons of concrete could have been provided. Thus the effective weight of wall and crane track provision was 110 tons per foot. Judged by British practice the wall would have been very light indeed. *Fig. 5* showed a design suggested by the Admiralty for the new dock at Cape Town. There the wall was built in "open cut"; good back fill was available and could be tipped to form ground shuttering for the cantiliver portion. At a depth of 71 feet, the weight (which allowed for the crane) was 115 tons per foot and, although that was heavier than shown in *Fig. 4,* it would be a cheaper wall because of the ease with which it could be constructed. Weight, therefore, was not the only criterion.

At Devonport, the wall had to be built in trench under very restricted site conditions in the heart of an active dockyard. With the design adopted each operation was complete in itself; excavations, when taken out, could be removed right off the site (no storage for back filling was required); when the concrete was placed the wall was finished: there was no fear of derrick cranes overturning into unstable back fill and subsequent 2 inch-driving plant for a piled crane track was not required. Under those very restricted site conditions, the wall was undoubtedly of the right type. By comparison with *Fig. 5,* it was 11 tons per foot too heavy, but the front profile was less favourable and, as " open cut " was not possible, the weight would not have been reduced below 120 tons per foot. The wall was designed to act independently of the floor—quite permissible in rock—but on the site a skew-back joint was made between the two.

Dam.—From the Admiralty point of view, the dam was good. It was completed within 6 months of placing the contract; it was almost completely watertight and had a reassuring appearance of ample strength. Final removal took 6 months, but sufficient was removed in 2 months to allow a ship to be docked. Removal was difficult because of the uneven quality of the underwater concrete, and the Author doubted if that defect would ever be overcome. Indeed, he doubted if concrete deposited in the dry (such as the old dock wall) could be blasted away in even regular layers 50 feet high. The long tube was designed to be used as a skip and would have had to be altered to be used as a tremie.

The Devonport dam was founded on good rock and the water-level in the closed basin was almost constant; there was no tidal range. Damage from the blasting was nil, because the size of charges was limited by the Admiralty. Small charges were always used first and were increased gradually to what was considered to be an acceptable maximum, arrived at only by " judging " the magnitude and effect of the tremors produced in the dockside area.

South End Wall.—The area between the south end wall and the existing caisson was only drained by the Admiralty because the caisson happened to have a through drain in it. At the north end, the caisson had no through drain.

Mr. Wilson's calculation of 2,400 cubic yards of concrete being placed in a volume of 3,600 cubic yards was correct on the information given in the Paper. By amplifying that information, however, the figures should be 3,000 cubic yards.

The actual concreting time was more nearly 7 days than 6; the skips were piled very full with dry concrete and contained " very good measure "; a period of very low spring tide was chosen, and in *Fig.* 7 the mean tidal range as given should be increased to nearly 20 feet.

Masonry.—Joints for dressed work were $\frac{1}{4}$ inch thick and the accuracy of 0·004 inch was quite adequate; but for future work it was most probable that concrete to an accuracy of $\frac{1}{8}$ inch would be used in conjunction with rubber gaskets on the caisson. Skip caissons were adopted because, with the widening of the entrance, there was not sufficient space between the adjoining docks to build cambers for sliding caissons. The sill arch had to be completely rebuilt on account of the increased span. Hardwood wedges were permitted up the quoins because they were only temporary; they were not referred to in the specification.

Venting of Floor.—Although the non-return valves on the vents did not work, owing to fouling of the seatings, some of the vents definitely weeped water when the dock was dry, and without them full hydraulc pressure would undoubtedly be built up. No attempt was made in the design to assess the quantity of water to be dealt with. In fact, it was very small indeed—probably no more than a few gallons per minute throughout the whole dock. Admiralty experience, however, at numerous dockyards at home and abroad, was that the danger of floor lifting was very real indeed and some floors had actually lifted. Quite small holes had always been sufficient to relieve the pressure and the Author concurred with the suggestion that floors should be of regular permeability throughout their full depth. Admiralty troubles had always been associated with the combination of an almost impermeable top layer of granite bedded on a relatively permeable, and much thicker, bottom layer of concrete.

Crane Track.—No attempt was made to drain the duplex tracks. Doubtless Mr. Ridehalgh's question was based on his tropical experience, where drainage was essential as an anti-malarial measure.

The crane track had to carry four wheel loads, each of 50 tons, spaced at 3-foot 6-inch centres, and the design was based on the usual assumption of a continuous beam on unyielding supports. On that basis, piles had to be at 6-foot 6-inch centres to keep the maximum pile loads to 100 tons. Since then the Admiralty had developed a method of taking into account the (relatively) very high stiffness of the beams and the settlement of the supports (the piles),[1] whence pile centres could be increased to 10 feet.

Dimensions.—The new dimensions, given as widening from 121 feet to 151 feet at barrel cope and from 74 feet to 103 feet at floor, referred, inadvertently, to lines used for setting out purposes. They should have been 121 feet to 143 feet and 74 feet to 114 feet respectively.

Pumphouse.—The pumps, as originally installed in 1906, were steam and were replaced during the widening of No. 10 dock by new electric pumps. The steam pumps were due for renewal irrespective of the widening but, on account of the latter, the new pumps were made larger.

Costs.—The costs referred to the civil engineering works only and did not include the caisson, the pumps, the cranes, or any other mechanical equipment.

In reply to Mr. Hooley, the small quantity of water running into the trial pits was dealt with in the open trench by simply running it into a channel. As the wall was built up the sump was also built up, and when the wall was halfway to the top and the sump was about 40 feet deep the water was pumped out very rapidly and the sump was filled with concrete as quickly as possible. The water had made the rock very greasy, and at the place where 30 feet of rock was unsupported a slight slip had occurred.

[1] See W. Elsby, "Continuous Beams on Elastic Supports," *Concrete Constr. Engng*, November 1943.

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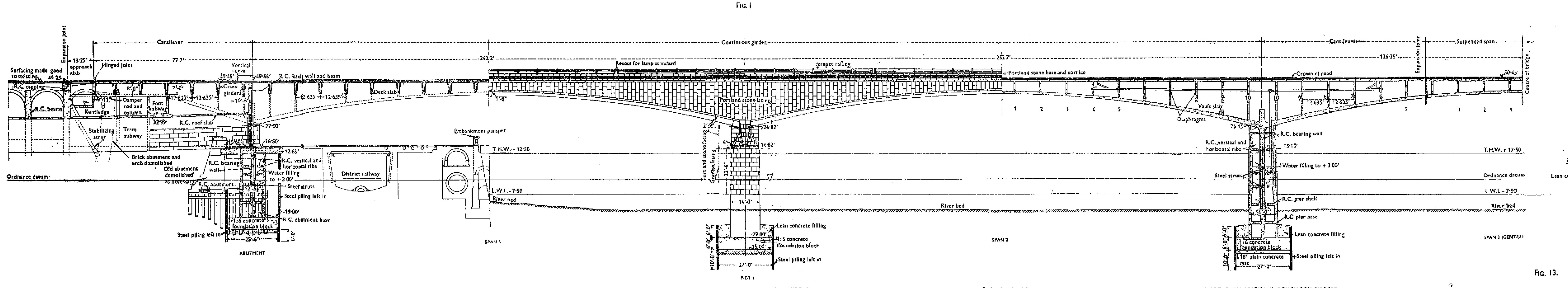
TEMPORARY WORKS AND CONSTRUCTIONAL DEVICES USED IN CONNEXION WITH THE CONSTRUCTION OF THE NEW WATERLOO BRIDGE.

PLATE I.
THE NEW WATERLOO BRIDGE:
TEMPORARY WORKS.

Fig. 1



LONGITUDINAL SECTION ON ℄ OF BRIDGE

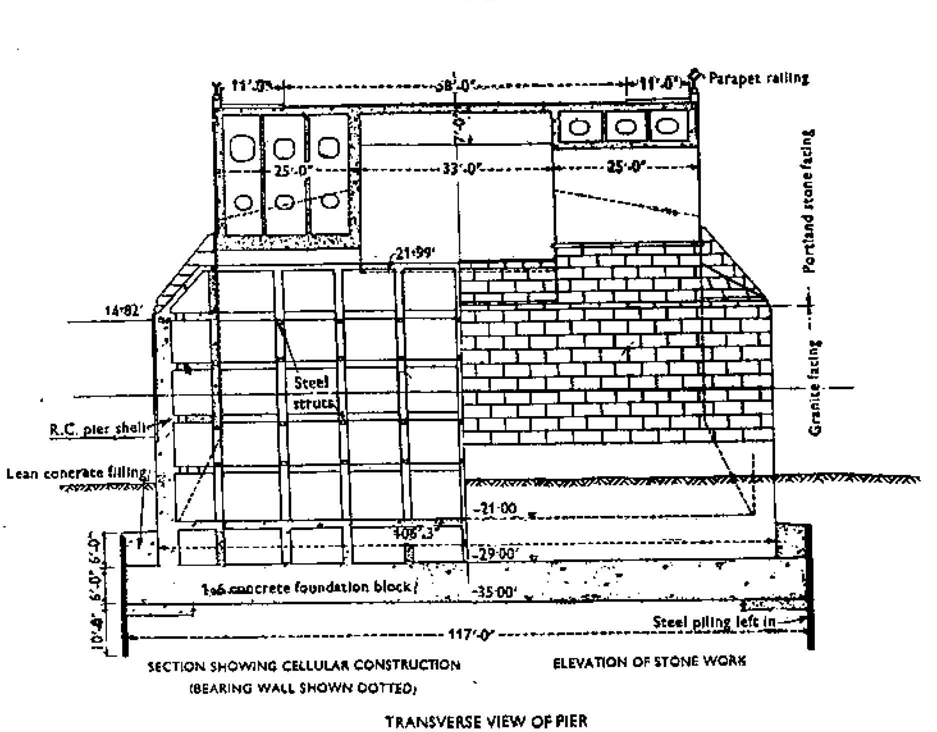
ELEVATION OF PORTLAND STONE AND GRANITE FACING

Scale 1/8 inch = 1 foot

LONGITUDINAL SECTION THROUGH BOX GIRDERS

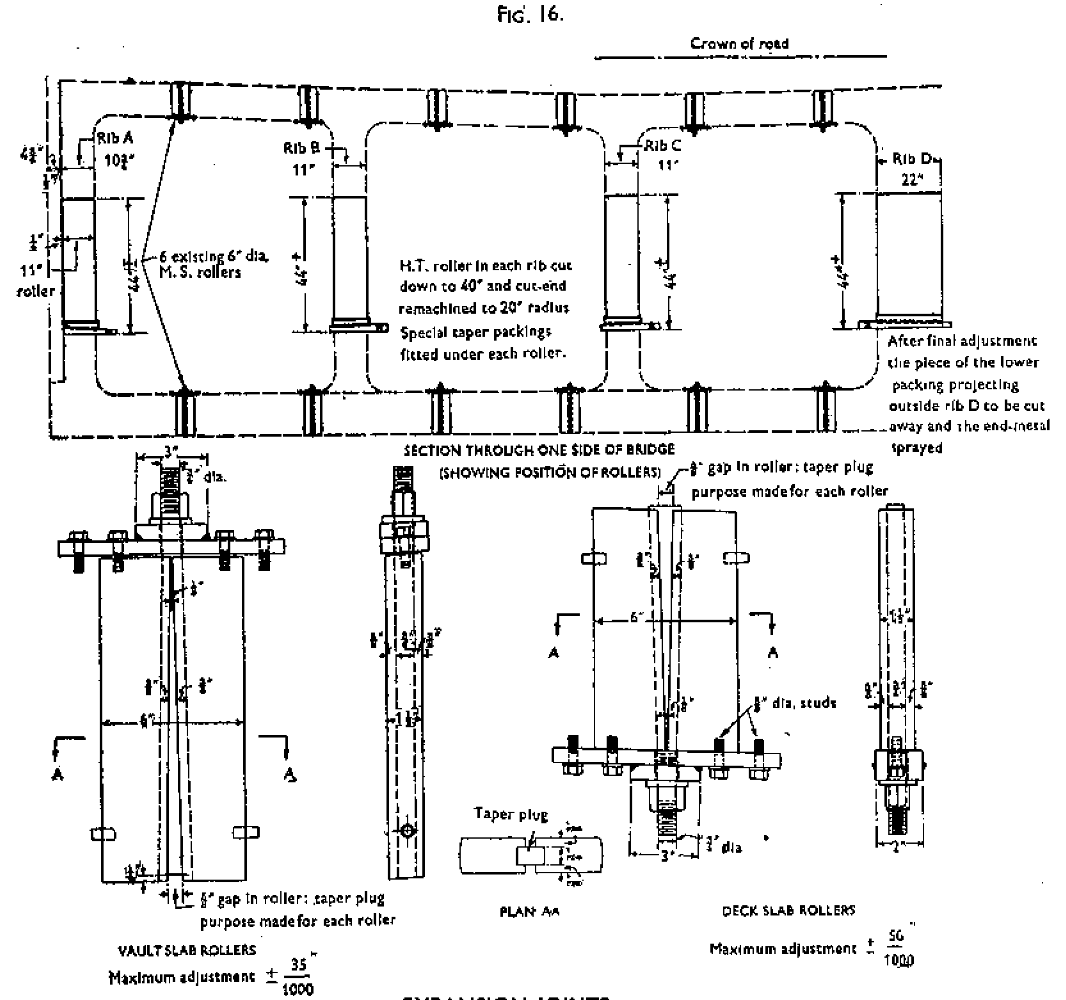
ELEVATION AND LONGITUDINAL SECTIONS THROUGH THE NEW WATERLOO BRIDGE

Fig. 2



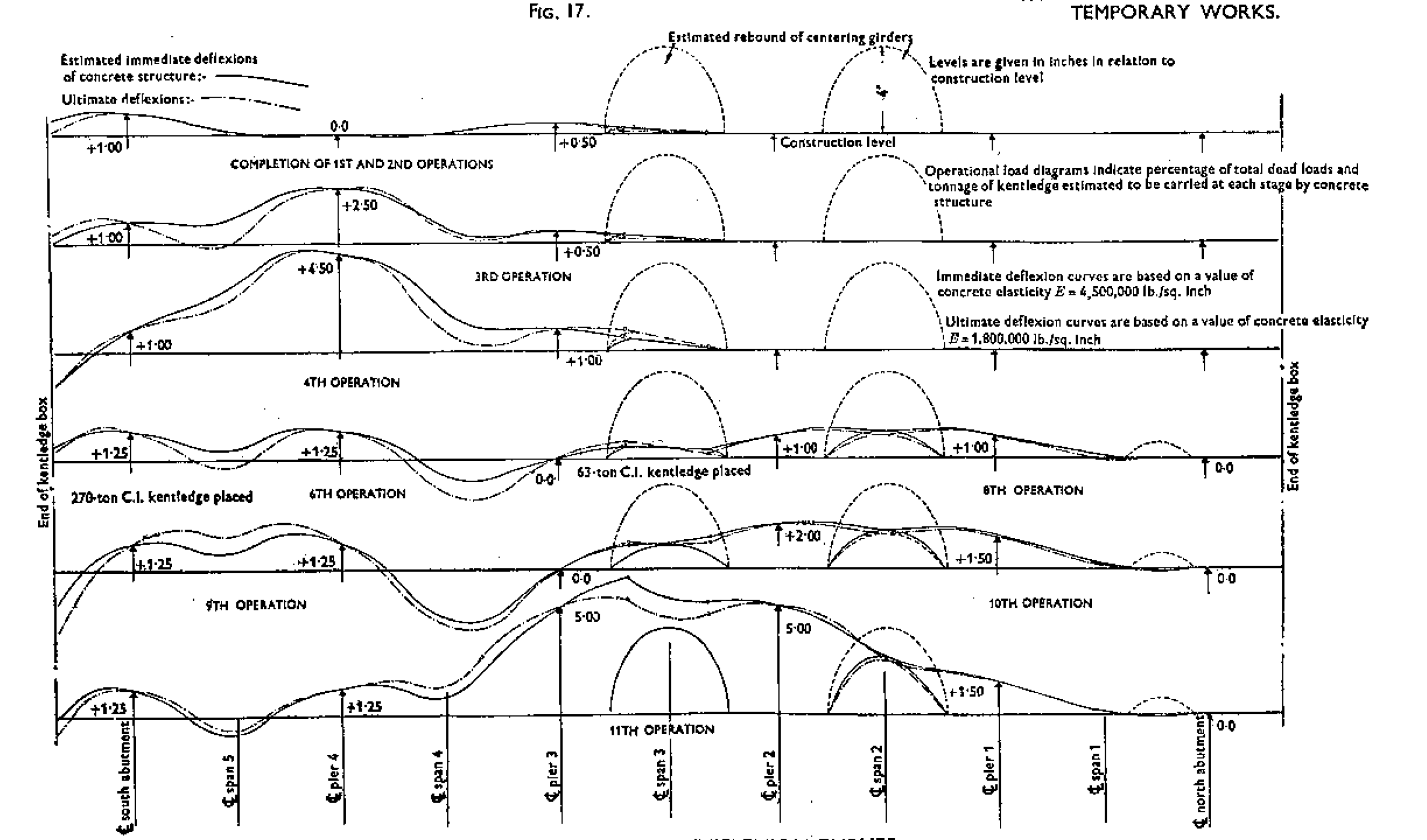
TRANSVERSE VIEW OF PIER

Fig. 16.



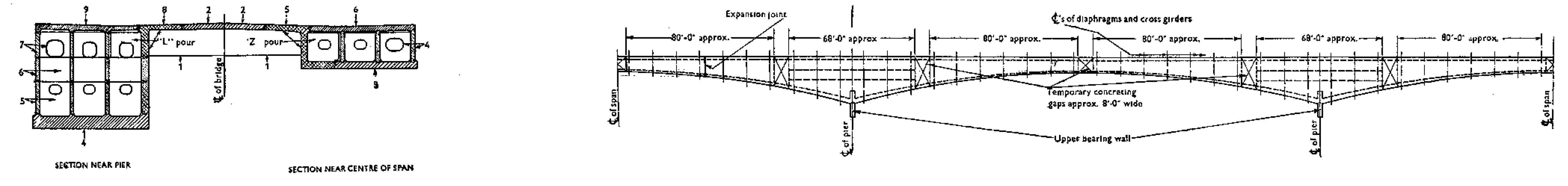
EXPANSION JOINTS

Fig. 17.



JACKING OPERATIONS: DEFLEXION CURVES

Figs. 8.

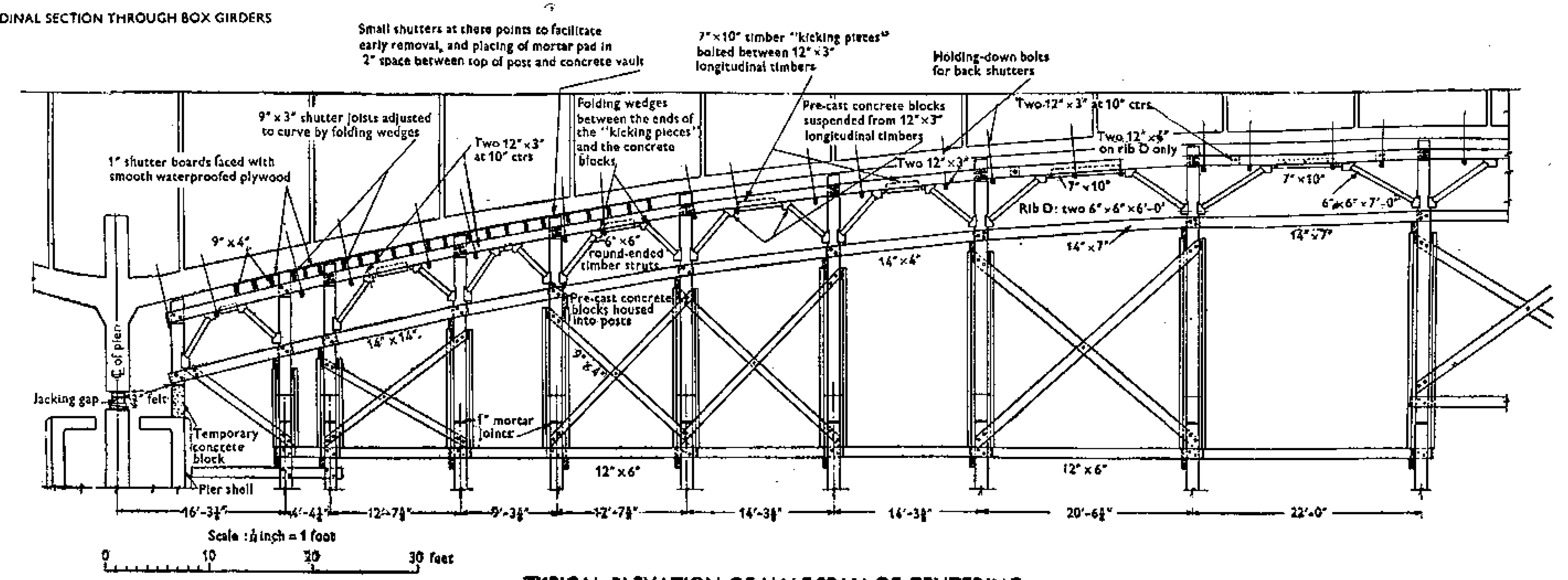


SECTION NEAR PIER

SECTION NEAR CENTRE OF SPAN

DIAGRAMS SHOWING SEQUENCE OF OPERATIONS FOR CONCRETING SUPERSTRUCTURE.

Fig. 13.



TYPICAL ELEVATION OF HALF-SPAN OF CENTERING

William Clowes & Sons, Limited : London.

The Institution of Civil Engineers. Works Construction Paper No. 9. 1947.

P. R. ALDERMAN AND F. E. PRIOR.

CORRIGENDA.

p. 31. Fourth formula. *For* "$\dfrac{Q}{Q^{\frac{1}{3}}/g^{\frac{2}{3}}}$" *read* "$\dfrac{A}{Q^{\frac{1}{3}}/g^{\frac{2}{3}}}$".

p. 52. Line 4. *For* "(ν_g)" *read* "(νg)".

p. 53. Formula K. *For* "$\alpha_9\left(\dfrac{XV_s}{(gD)^{\frac{1}{3}}}\right)^{\frac{1}{3}}$" *read* "$\alpha_9\left(\dfrac{XV_s}{(g\nu)^{\frac{1}{3}}}\right)^{\frac{4}{3}}$"

p. 53. Last formula but one. *For* "$\alpha\dfrac{Q}{g^{\frac{1}{3}}\nu^{\frac{1}{3}}}$" *read* "$\alpha\dfrac{Q^{\frac{1}{3}}}{g^{\frac{1}{3}}\nu^{\frac{1}{3}}}$".

All the formulae are for quartz sand and water. If any other material or liquid is used, a correction is required in terms of $\dfrac{\sigma-\rho}{\rho}$, where σ denotes the specific gravity of the material and ρ denotes the specific gravity of the liquid.