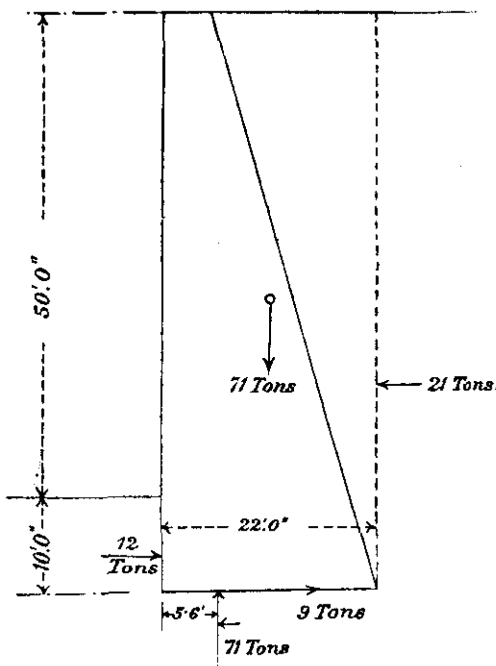
Correspondence.

Mr. Beare. Mr. E. W. Beare remarked that several very important points needed further discussion. First, the presence of water in the earth, behind, under, and in front of the wall; secondly, the question of friction at the back and at the toe of the wall, and thirdly, the position of the resultant of the earth-pressure under the wall. Mr. Wentworth-Sheilds's suggestion to calculate the lateral pressure of a flooded backing as that due to the earth alone, or else that of water whose free surface was at soakage level, whichever was the greater, appeared to be an excellent way out of the difficulty in most cases, but there seemed to be a touch of guesswork about it. It would probably not be far wrong for many materials, but with a very open backing it seemed that the lateral pressure would be greater than that due to water alone. In the case of a backing consisting of a granular material whose specific gravity was unity and which was flooded with water, the lateral pressure would presumably be that due to water alone. If a heavier granular material were substituted for the light material, it seemed that the lateral pressure would be thereby increased. Research was very badly wanted in that direction. If the earth under a wall was at all porous, the upward water-pressure should certainly be allowed for. It had the effect of lightening the wall and threw the resultant earth-pressure farther away from the centre of the base. The intensity of pressure under the toe might thus be increased, possibly beyond the limits of safety. With regard to the presence of water in the earth in front of the toe, the importance of research in this direction could not be too strongly emphasized. The permissible resistance of a flooded material might differ considerably from that of a dry material. In his opinion it was totally wrong to consider the friction between the backing and the back of the wall, particularly in the case of a large wall. The friction could only come into play if the wall tried to overturn about its toe. In the case of a large wall that was practically impossible, for a wall would not overturn about the toe unless the resultant of the base pressure was at the toe, and the intensity of pressure would then be too high for either earth or masonry. The inclusion of the back friction therefore presupposed unsafe design. It might be that there were

Mr. Beare. quay-walls standing to-day whose stability it was difficult to account for without taking into account back friction, but this was no reason for assuming that such friction was always present. It could not act if there were no tendency for the back of the wall to rise, and its value was absolutely indeterminate unless movement was about to occur, or was actually taking place. When considering the friction between the toe and the earth in front of it, there was no doubt that, if a wall sank at its toe, the friction would be called into play, but not until the ultimate resistance of the earth under the toe had been reached. Again, the inclusion of friction presupposed unsafe design. The usual assumption that the resultant pressure must lie within the middle third, was, he thought, open to question, for it was quite possible to have compression at the heel even when the resultant was outside the middle third. The common theory was that, when the resultant lay within the middle third, the pressure was distributed over the base, varying from a maximum at the toe to zero at the heel, and that, if the resultant lay outside the middle third, the pressure did not extend so far as the heel. If the designer of a wall found that the resultant came outside the middle third, he usually corrected the section of the wall because of the necessary minimum heel pressure which would tend to overturn it. He probably looked upon that as an extra force not already accounted for, and therefore unbalanced: hence

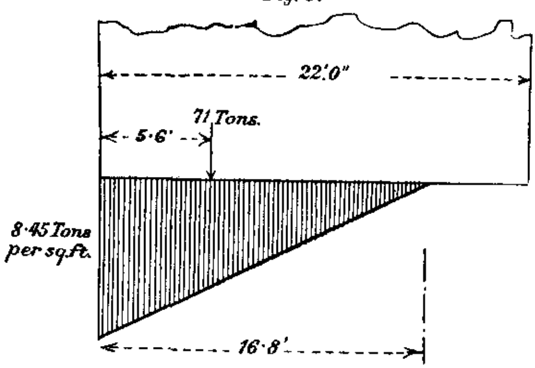
Fig. 4.



$\phi = 40^\circ$; $w = 120$ lbs. per cubic foot;
Weight of Masonry = 120 lbs. per cubic foot.

Mr. Beare. the revision of his calculations. As a matter of fact, the only effect of the heel pressure was slightly to increase the toe pressure, and if that was still definitely less than the permissible toe pressure, the wall would be stable against overturning. As an illustration he took the wall shown in *Fig. 4*. The magnitudes of the forces involved would be found to be approximately as shown in the figure. The resultant of the base pressure would be found to be 5·6 feet from the toe. The diagram of base pressures was usually taken to be as in *Fig. 5*, the maximum intensity of pressure being 8·45 tons per square foot—only about three-quarters of the maximum permissible load. A design giving such a diagram of base pressures was usually considered to be unstable, because it had not taken into account the fact that the backing was bound to produce an upward pressure

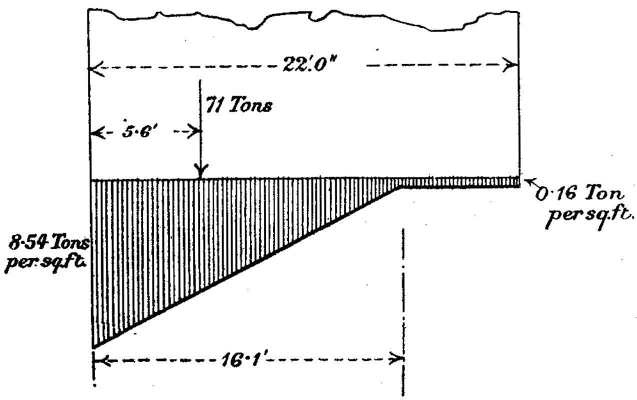
Fig. 5.



at the heel. That pressure did not by any means imply that the wall was unstable, for the diagram in *Fig. 5* was quite wrong. The true state of affairs was more likely to be as shown in *Fig. 6*. There was now a perfect balance of forces, and the heel pressure was sufficient to resist the upward lift due to the weight of the backing. It would be noticed that, by allowing for the necessary heel pressure, the toe pressure had only been increased from 8·45 to 8·54 tons, and was still well under the safe load. Such a wall would obviously not overturn. In the paragraph "Devices for Stabilizing Walls," Mr. Wentworth-Sheilds said that, theoretically, deepening the foundation of a wall might increase its tendency to overturn. Mr. Beare would like him to give particulars of such a case.

Mr. Bell. Mr. A. L. Bell remarked that unfortunately in most, if not all, of the numerous examples of failures of dock-walls, the necessary data for calculating the forces which caused the failures were not available. It would add greatly to engineering knowledge, and would have a beneficial effect upon practice, if a standing committee were set up by The Institution to secure precise particulars of the nature of the soil and other necessary data in future cases of failure. He had, at considerable pains, examined a large number of publications dealing with foundation loads, failures of retaining-walls and other structures, skin-friction, and other points bearing upon the problems of earth-pressure and resistance, and had found, to his disappointment, that no precise conclusions could be drawn from them. In most cases something essential was missing, e.g., foundation-loads Mr. Bell.

Fig. 6.



were given without reference to the depth, skin-friction without sufficient assurance that the cutting edge was free from support, and so forth. A committee, though it might not be able to prescribe formulas for calculation which would meet with general approval, could at least put all the essential facts upon record. In the course of time very valuable particulars might be collected, the cumulative effect of which would be great; and time would doubtless show, by practical examples, which formulas were safe and which were not. In another respect such a committee would be of value. At the present time, no doubt, many engineers were, without collaboration, testing foundations by various methods of their own devising. If details of those tests could be placed, as a matter of routine, at the disposal of a committee and subjected by the latter to expert analysis and tabulation, mutual benefit might result.

Mr. Bell. A large number of such tests might ultimately show the true values of the characteristics which were required when applying the available formulas for the yielding point of foundations, namely, ϕ for dry sandy foundations, or k and a for foundations in coherent earth. Up to the present no direct and entirely satisfactory method of determining those characteristics had been devised. With regard to the modes of failure of retaining-walls, it should not be overlooked that failure might take place, not in the material upon which the wall was founded, but in softer material lower down.[1] Many cases had occurred in his own experience where, at a certain depth, excellent founding material had been met, but, owing to the existence of softer strata underneath, it had been necessary to carry the foundation much deeper, in order to avoid the risk of failure from that cause. Mr. Wentworth-Sheilds referred to failure arising from the turning of the wall upon the toe accompanied by rising of the heel. It was questionable whether that ever happened. The failure of the material under the toe must be the immediate cause of overturning in all cases where the wall was not founded on rock. He had no doubt that, where the material was uniform, the expedient of sloping the foundation upwards from heel to toe gave additional security against sliding forward, for the wall must then either slide upwards (which was unlikely) or shear the undisturbed material horizontally; and more force was required to shear the undisturbed material than would be required to push the wall along a previously prepared horizontal surface. As for the proper procedure to adopt when calculating the stability of a wall backed by a mixture of water and earth, the information now available was inadequate, and further experiments upon the point were required.

Mr. Best. Mr. A. T. Best, commenting upon Mr. Wentworth-Sheilds's Paper, remarked that the subject was one of perennial interest, and the Paper was valuable chiefly by reason of its clear statement of the problem and its factors. Some of them were unknown quantities, and the problem might be incapable of solution in exact terms. The Author, however, invited the pooling of information. Mr. Best's remarks were therefore offered as the outcome of a certain amount of experience gained in connection with the detailed calculation, design, and construction of heavy quay-walls of the character under review. Although the Paper opened with a reference to "strength and stability," it was thereafter concerned almost solely

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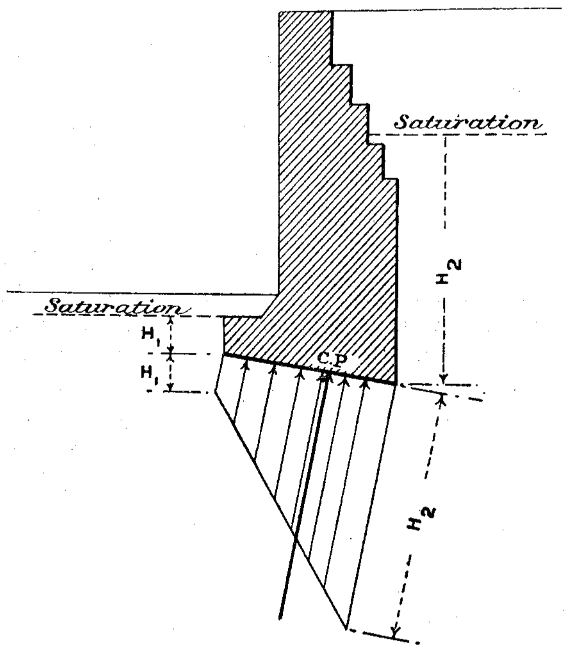
[1] See remarks by the late L. F. Vernon Harcourt, Minutes of Proceedings Inst. C.E., vol. xxi, p. 120.

Mr. Best.

with stability. His comments were therefore confined likewise to stability, or resistance to bodily movement, as distinct from strength, or resistance to deformation and fracture. Considering the stability of a wall under the pressure of external forces tending either to move it or to keep it still, thirteen forces were enumerated, apart from the weight of the wall itself. Some of those were usually "neglected" in calculations, but that could only be done with impunity if they were really of negligible amount; for it might be taken as an axiom that Nature neglected nothing. Fortunately, more than half of the thirteen were friendly, engaged in preserving the *status quo*. The factors of most interest, combining great influence with great uncertainty, were those of the principal earth- and water-pressures, enumerated as A1, B1, and D1. Regarding A1 (lateral pressure of backing), Rankine's formula, quoted as $P = \frac{wh^2}{2}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$, might be more briefly and conveniently stated as $P = \frac{wh^2}{2}\tan^2\theta$, where $\theta = \frac{1}{2}(90° - \phi)$, and within ordinary limits this could be shown to coincide with the sliding-wedge theory. Another important point to be realized was that this theory, whether employed graphically or analytically, applied to fluids and semi-fluids as well as to granular substances such as earth. Water was no exception to the rule, but responded to exactly the same method of calculation, the angle of repose ϕ in that case being zero. Then $P = \frac{wh^2}{2}\tan^2 45°$, and as $\tan 45°$ was unity, therefore $P = \frac{wh^2}{2}$, as universally recognized for water-pressure on a vertical face. Such conformity with law helped to the right estimation of pressures due to saturated or liquid substances. The greatest possible lateral thrust of backing on retaining-walls was due in his belief to mud or dredged silt having a unit weight of about 128 lbs. per cubic foot, and an angle of repose of 15°, or 1 in 4. (In another line of inquiry, the pressure of fluid concrete when deposited against shuttering was still greater.) Taking the pressure of water as 100, that of mud on the above data would be 120, while that of dry earth weighing 112 lbs. per cubic foot, with $\phi = 30°$ would be only 60. The case of saturated granular soil, however, was one of special difficulty. He questioned the adequacy of Mr. Wentworth-Sheilds's proposal to take the earth-pressure or the water-pressure, whichever might be greater. A better practice might be to take the water-pressure plus that due to the weight

Mr. Best. of a sliding wedge of earth, but with only the upper or dry portion of the wedge at full unit weight, that of the lower or wet portion being reckoned at a reduced weight (as weighed in water) allowing for the effect of flotation, which varied with the percentage of interstices. That assumption was more severe and consequently safer than either of Mr. Wentworth-Sheilds's proposals, but it might still be inadequate, as it took no account of any flattening of the angle of repose caused by the presence of water regarded as a lubricant. The Author's proposal to consider B1 (pressure of water in front) at lowest tide-level should be followed with caution. Behind a wall in a tideway the saturation-level probably remained nearly stationary about midway between high water and low water, owing to the slow rate of percolation. Thus at low water the water-pressure on the face would be less than on the back. In an enclosed dock the pressure would always be greater on the face of the quay-walls and would tend to increase stability after the dock was filled. It appeared imprudent, however, to base calculations upon such aid, because during construction it was non-existent. There was no water in front of the wall to balance the pressure not only of the earth filling against the back but also of saturation water rising behind it after completion and before flooding. In such conditions Mr. Best had observed water oozing through from back to face throughout a long length of quay-wall, up to about half its height above the dock-bottom. With regard to B2 (lateral resistance of earth in front of the toe) the Author had done a signal service in emphasizing the difference between the active pressure and the passive resistance of various materials. The resistance was at a maximum and the pressure nil in the case of a perfect solid. With liquids the reverse was the case, the pressure being great and the resistance slight. Semi-fluids and granular materials lay between the two extremes. A factor which had very great influence, although sometimes it was overlooked, was the upward pressure of water under the base (D1). It was necessary, however, to challenge the reasonableness of taking that as uniform and equal to the head of the lowest water-level. From observation of trench-bottoms, he was convinced that in loose ballast the saturation was complete and circulation or percolation was sufficiently free to ensure that the hydrostatic upward pressure at the toe and heel equalled the head at the front and back respectively, and probably varied evenly between the two points. From that there followed the important conclusion that, as the pressure was not uniform, the centre of pressure was not coincident with the centre of the base. Consequently, under a dock wall after flooding, with the head of impounded water on the face Mr. Best. exceeding the head of saturation water behind, the upward pressure was nearer to the toe, and resisted overturning. But before flooding with ground-water at the back only, the ordinates of pressure must decrease from back to front as shown in *Fig. 7*. The centre of pressure came nearer the heel and created an overturning moment about the toe, thus increasing the element of danger previously mentioned which arose during construction. On the other hand, after water had been admitted, its presence on the face introduced

Fig. 7.



the factor of buoyancy or partial flotation by increasing the upward lift under the base. That virtually reduced the weight of the wall and consequently the friction on the base, which was a function of the weight. Thereby the resistance to sliding was affected. Thus, consideration of the influence of water, so rightly brought to the fore by the Author, was fraught with complications. To ensure security, it was necessary to calculate stability under various conditions which obtained at different stages of construction as well as after completion With regard to D3 (upward resistance of the earth)

Mr. Best. it was noteworthy that the Author still considered the "middle third" rule a wise one, at least in regard to the base of a wall. The essential point to watch, however, was not whether the centre of resultant pressure kept within that limit, but whether the intensity of pressure upon the soil at the toe was kept below the permissible maximum. The best way to effect that was by "a longer toe" as mentioned approvingly elsewhere in the Paper and as adopted by the Author at the White Star Dock, Southampton.[1] The "middle third" principle, if applied to the body of a wall, led to an uneconomical cross section through excessive weight, for in general the resultant might be allowed nearer to the face without passing the margin of safety. Pursuing that line of argument, it was even demonstrable that the stability of a wall might be increased by the paradoxical method of reducing its cross-sectional area, provided such reduction were made in the right place and the internal strength were maintained. Although stability and strength were closely allied, the latter aspect of the matter might well form the subject of a separate investigation. He therefore hoped to return to it by the submission to The Institution of a Paper thereon.

Mr. Buckton. Mr. E. J. Buckton thought that Mr. Wentworth-Sheilds's Paper gave a most clear and useful outline of quay-wall design. The opening remark "that many engineers discard their calculations, even if they have made them, and trust to their judgment in deciding whether a wall as designed will be stable," was rather startling if it was meant to apply to deep-water quay-walls. It was hardly conceivable that an engineer would design an important and costly wall based solely on his judgment. In the case of a flooded backing the pressure must be equal to the water-pressure, plus a pressure due to the earth backing taken at its displaced weight; but the value of ϕ under water might, and in many cases must, be different from ϕ for the same material in the dry. Further knowledge of the value of ϕ for materials when submerged would be useful. There could be no rule as to what superload should be allowed for quays, as the amount depended entirely upon the local conditions. Many quays had railway-lines between the quay-wall and sheds, which could not be carried on piles; and, if the lines were within the surface area of the earth retained by the wall, the weight of the locomotives must be taken into account in finding the superload. The foundations of the buildings could usually be designed so as not to affect the pressure on the wall. The crane rails also could generally be arranged so that, if any load came on the

[1] Minutes of Proceedings Inst. C.E., vol. cxcv (1914), p. 42.

Mr. Buckton.

wall, it tended to increase its stability. A good example of such a case was where the outer rail ran along the top of the wall, vertically over the middle third of the base, and the inner rail was carried on a beam supported by piers resting on the steps at the back of the wall. It was no doubt good practice, and it was much the simplest way, to take the superload as uniformly distributed, but the amount mentioned, 2 cwt. per square foot, had no general application. In the case of a long quay-wall at present under construction a superload of 12 cwt. per square foot had been taken. Knowing the arrangement of cranes and railway-lines, and the nature of and method of handling the goods to be loaded and unloaded, it was a simple matter to decide upon a suitable superload. It seemed more reasonable to base the length of quay, over which the pull of a ship's moorings was assumed to be distributed, upon the height of the wall, rather than on the thickness at the base. If vertical settlement-joints were not formed in a mass-concrete wall during construction, irregular cracks, running approximately vertically from top to bottom of the wall, usually occurred at intervals soon after completion. If settlement-joints were formed at intervals equal to four times the height of the wall on good foundations, or twice the height of the wall on poor foundations, settlement-cracks seldom appeared except at the joints provided. It was therefore reasonable to assume that the pull or pulls could be taken as distributed over a length of quay equal to two to four times the height of the wall, according to the quality of the foundations. The relative importance of the effect of moorings on the stability of an ordinary quay-wall was small, and was often ignored. The conditions with a flooded backing applied to the resistance of earth in front of the toe, except that in the latter case the resistance was passive, and in the former case active. There seemed little reason to doubt the advisability of giving a small slope to the base of the wall, say 1 in 10 to 1 in 20, as for the same maximum depth of foundation there was greater resistance to horizontal movement, the shearing resistance of the earth beneath it in a horizontal plane being almost certainly greater than the friction between the wall and the earth, while the passive resistance of the earth was the same. A slope to the base not only had the advantage of increasing the resistance to sliding, but it also saved concrete, and provided better drainage of the foundations during construction. Any friction between the backing and the wall assisted in preventing overturning, but in a quay-wall it was good practice to ignore it, as the friction in a flooded backing was a small and uncertain quantity. It was safest to take the pressure at the toe of the base of a wall as that due to a head of

Mr. Buckton. water equal to the height of the water in front of the wall, and at the heel as that due to a probable head of standing water behind the wall, the pressure between the toe and the heel being uniformly graded. Due to partial impermeability of the material under the base, the upward water-pressure might be less, but it was not a dependable condition, and should be ignored. No doubt vertical frictional resistance existed between the toe and the earth in front of it, but it was best neglected in practical designing. The pressure at the heel should not be less than any upward pressure caused by the backing tending to upheave the earth under the wall. It was pointed out in the Paper that the upward earth-pressure was possibly neutralized. Where the foundations were in good, firm, undisturbed ground, the tendency of the ground to upheave at the heel could be neglected, and the resultant might be allowed to pass slightly outside the middle third, always provided the resultant maximum compression at the toe was within the safe bearing-power of the soil; but where the foundations were poor, or the new backing was carried right down to the heel, the resultant should be kept within the middle third. The safe bearing-power of the soil must be found experimentally on the site. Naturally, for the sake of safety a somewhat lower maximum value was adopted than the maximum found by experiment, but usually no large factor of safety was necessary. An ordinary mass-concrete wall with deep foundations was not more likely to overturn than was one with shallower foundations, but to go beyond a certain depth was uneconomical, as the wall would fracture, due to excessive tension at the back, and it would be left standing on expensive concrete where the earth might be sufficiently firm for the purpose. In the old days the practice of stepping the back of a wall was the most convenient way of altering its section gradually, but, with the present practice of using mass-concrete, there was no reason why economy should not be effected by using a batter instead of steps. It was still the practice to give a slight batter to the face of a wall for the sake of appearances. An appreciable batter was a distinct disadvantage in a quay-wall used by modern ships, as ships now had a greater beam below water than above, and for convenient working it must be possible to bring them close alongside the quay. A toe was usually adopted, as it gave a more economical section near the base, but the size was limited in practice, as it must not interfere with ships alongside; and if, in order to leave the water-space unrestricted by any projection, it was made long and shallow by using reinforcement, excessive tension would be set up in the mass concrete at the back of the horizontal section immediately above the toe. A tensile stress of 14 lbs. per square inch was sometimes allowed in designing a quay-wall. That was important, as a small tension allowance considerably reduced the thickness of the wall, with a corresponding reduction of concrete. Although it was better to assume that the concrete, particularly between the layers as deposited, had no real tensile strength, it was good practice to make an allowance for tension when designing a wall, as the tension at any horizontal section was a measure of the amount the resultant at that section was outside the middle third. As concrete had a high crushing-strength, there was no reason why the resultant should be confined to the middle third. If the resultant was outside the middle third and the concrete could take no tension, it merely meant that the width of base over which the resultant acted was three times the distance of the resultant from the face, and that the maximum unit pressure due to the resultant would be somewhat greater than if the concrete could take tension. The maximum unit pressure, however, might still be well within the safe crushing limit of the concrete. If, as was often the case, the possible upward water-pressure at any horizontal section were taken into account, it was certainly safe to allow tension in the concrete. Mr. Buckton.

Mr. A. E. Carey considered that the final court of appeal in connection with problems of the stability of river- and sea-walls must always be experience. Rankine's formula was the standard of theoretical reference with most engineers, but the data on which it was based were obviously inferential. In his experience a river-wall seldom gave way at the base or slid forward. Casualties were more frequently due to the bulging of the upper portion of the wall and consequent dislocation of the structure than to its movement as a whole. Theoretical considerations were based on the assumption of the interaction of a number of known or partially-known forces from without the structure. One difficulty of combining those data was the fact of the inequalities of effect produced by tidal action. It was obvious that the soakage due to land water or tidal penetration set up an infinitely varied series of pressures in proportion to the degree of permeability of the backing and its avidity for the retention of imprisoned water. The periodic variations of intensity of internal stresses due to that cause would probably be frequent factors in the disturbance of equilibrium. The behaviour of a clay wall under distorting stress was, he thought, a good object-lesson. In nine cases out of ten it would be found that slabs of such a wall near its crest sheared through water under pressure finding its way through fissures or lines of weakness. The result was a slip or slips, the bulk of the wall being undisturbed. If the slip was Mr. Carey.

Mr. Carey. sufficiently serious to permit of water flowing over the crest of the wall, the results were guttering and flooding in rear of the wall.

Mr. Latham's Paper raised a difficulty which experience had shown to be crucial. Mr. Carey had been an early advocate of deep-water quays on the Thames. At the time of the Port of London Authority Parliamentary Inquiry, evidence was produced in favour of such projects. The argument was that, geographically, big ships should be catered for where deep water existed naturally, thus obviating the great expense of constructing docks and producing deep water artificially. The nautical difficulty of holding a ship in a tideway constantly varying in height was a serious dilemma when it was sought to create a long line of quayage. An isolated berth caused no difficulty, as the ship's ropes could be carried ashore forward and aft of such a structure. The design of the Thames Haven jetty now under construction had been evolved by his firm in order to obviate risks to dolphins in manipulating big tankers. Such vessels came in at high water and swung and dropped alongside the jetty early on the ebb; and in a strong tideway they were difficult to control. While the principle of deep-water quayage could, he thought, be maintained, mooring vessels to an *intermediate floating structure* seemed to him the readiest solution of the problem.

Mr. Carron. Mr. F. G. CARRON had experimented with two model concrete walls, each 24 inches long by $11\frac{1}{2}$ inches high. One had a plain sloping back and the other had a stepped back. Each model weighed approximately 82 lbs. The models were placed in turn on a bench and caused to retain material contained in a frame, the front of which was formed by the model. Ropes fixed to the walls at one-third the height and at intervals of 6 inches, passed over pulleys and held pails which were filled with sand until the walls overturned. The results were :—

Material retained.	Overturning Force in Lbs.	
	Stepped Wall.	Sloping Wall.
Sand ($\frac{1}{4}$-inch gauge)	91·75	74·25
Loose dry earth	74·5	65·0

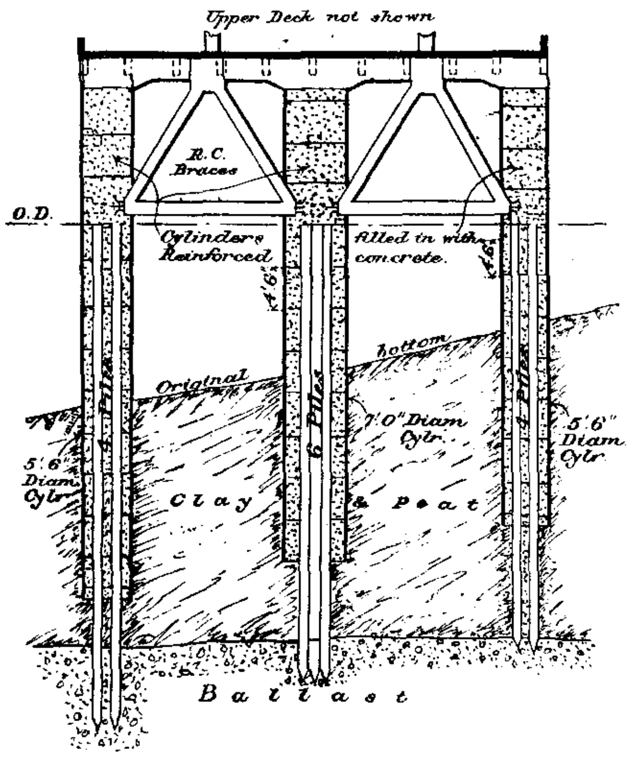
On checking by calculation the result in the first case, he found that the moment due to the total weight of sand supported by the off-sets almost exactly accounted for the difference in the overturning moments.

Mr. W. DYCE CAY thought that, in many cases, the system of building deep-water quays with concrete in bags or in mass might be found advantageous and certainly economical. With regard to the strength of such work, while extreme hardness of unset concrete deposited in water could not be attained without using a quantity of Portland cement at least equal in bulk to the sand employed in mixing, still experience showed that many parts of a work had not required that flint-like hardness; and in any case it was a matter which could be easily experimented on with the materials proposed to be used for any particular work. He mentioned two works he had constructed in that way. One was a quay-wall for fishing-boats at Aberdeen Harbour. The depth, prepared by dredging, was 6 feet below L.W.O.S.T. and the rise of tide was 12½ feet. Sheet piling along under the front of the quay, and bearing-piles under the base, of the exact length required, were driven by the use of a long timber dolly with cast-iron jaws. The bags were deposited on the piled base by a 12-ton hopper skip, and above low water the wall was built of mass concrete; the substratum of the site was alluvial. The other work was the steamboat pier at Lerwick. There the bottom was soft silt covering rock, sloping seaward in the direction of the pier. The silt was dredged and the bag walls were built from a barge lowering a 9-ton hopper skip to helmet divers. The depth was about 20 feet at low water at the outer end of the pier, with a tidal range of about 6 feet, and the top was finished with mass concrete. Greenheart sponson fenders were used. The works had been in use for many years. Mr. Cay.

Mr. F. M. DU-PLAT-TAYLOR had read Mr. Latham's Paper with much interest. He suggested that it would have been more appropriately entitled "Deep-Water Jetties," the term jetty, implying a structure "thrown out" into the river from the shore, being that generally used on the Thames. With regard to the cylinder type of jetty, the Author stated that "some difficulty has been experienced in constructing such cylinders in reinforced concrete." To that statement Mr. Du-Plat-Taylor ventured to demur, as in the case of the deep-water jetty at Tilbury no difficulty had been experienced in the making of the cylinders, which were moulded in sheet-steel moulds filled upon shaking tables, and very few had been rejected as defective. The Author mentioned two methods of fixing or sinking such cylinders, one being first to drive the piles and then place the cylinders over them, and the other to cast the cylindrical piers in position by the use of steel-sheath moulds. Neither of those methods had been adopted at Tilbury. There the reinforced-concrete cylinders were pre-cast, as described above, in Mr. Du-Plat-Taylor.

Mr. Du-Plat-Taylor. lengths of 4 feet 6 inches, those for the outer rows at the back and front of the jetty (*Fig. 8*) being 5 feet 6 inches in diameter, and those for the centre row 7 feet in diameter, the thickness being 4 inches in the small, and 5 inches in the large cylinders. The lowest ring of each pier was formed with a cutting edge (*Fig. 9*), making the lower edge of the cylinder 1½ inch larger in diameter than the body of the cylindrical pier. The cylinders were

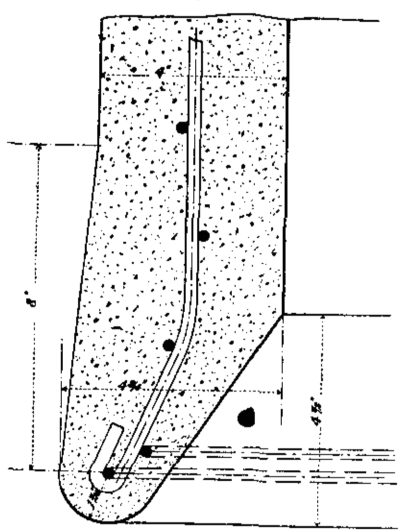
Fig. 8.



pitched in the bottom of the river, and were then sunk to the desired depth by grabbing in their interior by means of a special grab and steam crane. When the cutting edge had reached the required depth, groups of reinforced-concrete piles were driven within the cylinders, four piles in each 5-foot 6-inch pier and six piles in each 7-foot pier. To guide the piles and to prevent their fouling the interior surface of cylinders, mild-steel guide-cages were fitted on top of the top ring of the cylinders just above low-water level

(*Figs. 10* and *11*, pp. 176 and 177), and the piles were pitched through those guides. The act of releasing the 14-inch piles when inserted in the guides, and the consequent drop on to the river-bottom, sufficed to embed them 3 to 5 feet in the mud, and the driving to a depth of 2 feet into the underlying ballast was completed by an ordinary pile-driver. The steel-grid guide-frame ensured the piles being truly vertical and correctly spaced. When the pile-driving was completed in any pier, the interior of the cylinders was washed out by means of a water-jet, and any mud clinging to the inside of the cylinder was washed away over the top; the interior was then filled with mass concrete reinforced with vertical bars embedded around the heads of the piles, the bars being carried up into the braces and decking above. Very little difficulty was experienced in sinking the cylinders, the only obstacles encountered being portions of old tree-trunks, etc., in the layers of peat below the river-bed, and those were usually easily broken up by the grab. The pile-driving also presented no difficulty, which he attributed to the use of the pile guide-grids which he had suggested to the contractors. He considered that the cylinder-pier type of construction for river-jetties was superior to any other, both in ability to resist shocks and in the complete clothing of the supporting piles at the danger-point between wind and water. Mr. Du-Plat Taylor.

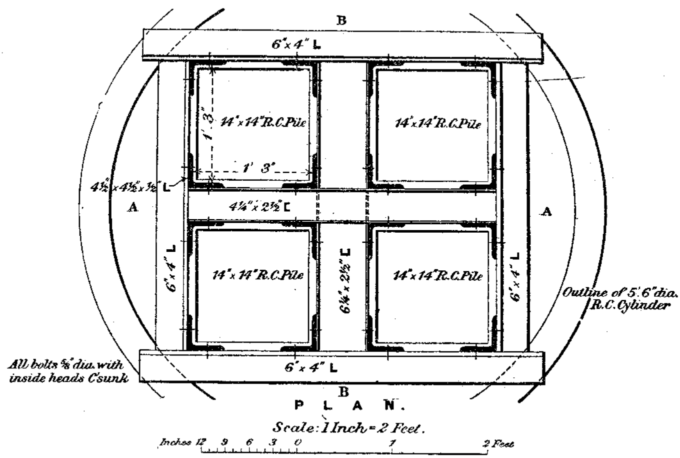
Fig. 9.



Mr. F. W. DUCKHAM considered that both Papers were particularly illuminating in showing the lack of finality which still existed within such a comparatively simple and frequent subject of engineering practice. Mr. Latham showed a well-considered plan for the No. 6 Quay at Thames Haven, with a corresponding cross section of a simplicity which was welcome when compared with many existing designs, whilst Mr. Wentworth-Sheilds brought up to date the considerations for stability in respect to all factors, except that of shock horizontally inwards, which certainly demanded consideration where there was no back filling—as in the case of any skeleton quay Mr. Duckham.

Mr. Duckham. such as the one illustrated by Mr. Latham. Mr. Duckham, however, was disappointed that there had, as yet, appeared no sign of consideration from the broader aspect, which had frequently been impressed upon him through physical experience, of the horizontal reaction of heavy staging to the influence of heavy seas and of shocks from vessels, combined sometimes with vertical loads of the heaviest character. The conclusion, which had been enforced by his experience, was that all such structures might best be visualized and considered as being vertical cantilevers embedded at their base. When such an image was realized, and the structure was considered as a narrow slice of the whole quay, it appeared quite evident that the heaviest bracing should be in the lowest panel, so as to resist the total shear,

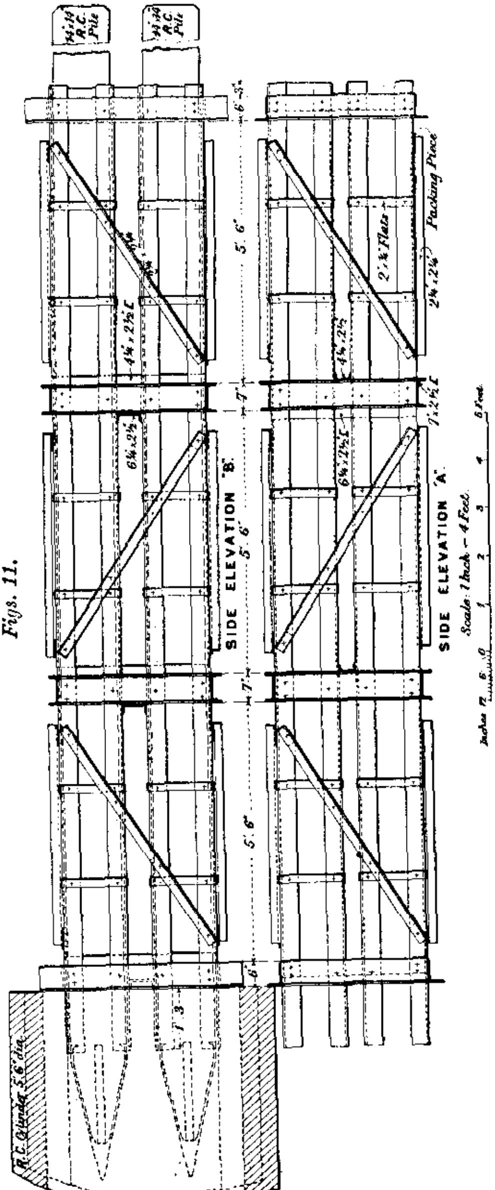
Fig. 10.



and also that any piling should be concentrated towards the two edges, which, back and front, corresponded with the two flanges of a beam. Similarly, if practicable, the lower portions of such piles should be stronger at the bottom panel than elsewhere. It thus followed that the rear piling would require special consideration and anchorage, in the case of a quay which had to resist a heavy backing, whilst an open wharf or quay of the type shown by Mr. Latham would, on the contrary, have to develop its greater resistance against the shock of vessels or of waves from the front. Here the front piles would clearly be subject to tension and even to drawing, whilst the back piles would be in simple compression, except during the inconsiderable occasions of the comparatively small pull by the mooring-ropes of a ship. In the light of that simple view, it became

Mr. Duckham.

palpable how much piling was often comparatively wasted within the middle of the cross-section in positions which would thus approach the imaginary neutral axis of the vertical cantilever, and where they had little function except to resist vertical loads. It further appeared how, by proper triangulation down to the bottom, the inside and outside piles ought to be caused to admit full resistance to simple tension or compression, rather than have merely to form the total of so many separate units of resistance against so many horizontal bending moments. This latter condition of design was only too often the case and was almost entirely so whenever all the bracing was confined to levels above low water, as was shown in Mr. Latham's section. Those piles certainly had a considerable advantage in being sheathed by cylinders, but their effect might be much improved by the

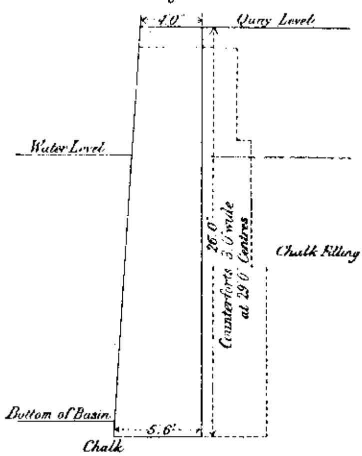


Figs. 11.

Mr. Duckham. addition of a simple diagonal tie from the bottom of the front cluster up to the junction at the top of the back piles. Mr. Duckham realized that such under-water bracing was avoided as much as possible, in order to obviate extra cost due to the work being done by divers. Long and special experience with such bracing had shown no need for extravagance when it was attached to steel clips and tightened by union screws. Thus the most effective panels of under-water bracing might be readily constructed right down to the root of the piles, and the highest efficiency obtained thereby.

Mr. Hollingworth. Mr. E. W. HOLLINGWORTH considered that the economical design of retaining-walls was perhaps the most important engineering problem remaining unsolved, for the consequences of failure were so serious that few engineers cared to take any risk, and, as all were not gifted with the "miller's thumb," vast sums of money had been spent in excessively heavy work. Of the many factors enumerated by Mr. Wentworth-Sheilds, the properties of the material in the foundations and backing were the most important, and so many assumptions had to be made concerning them that Sir Benjamin Baker had said it was as well to assume the thickness of the wall. The rule that the centre of pressure should fall within the middle third of the base tacitly assumed that the soil was perfectly elastic, which fortunately was not the case; few dock-walls complied with that requirement. Departure from the elastic condition increased ultimate resistance by enabling less-stressed parts to take a larger share of the load, and this had enabled a hook of mild steel to withstand repeated applications of forces which would cause a fibre stress of nearly 40 tons per square inch in perfectly elastic material. The properties of soils were so complex that there seemed little prospect of formulating rules to take the place of judgment and experience, but as individual experience must be limited, the proposed collection of information by The Institution would be invaluable. In that connection the North Wall of Ramsgate Harbour might be of interest (*Fig. 12*). The factor of safety must

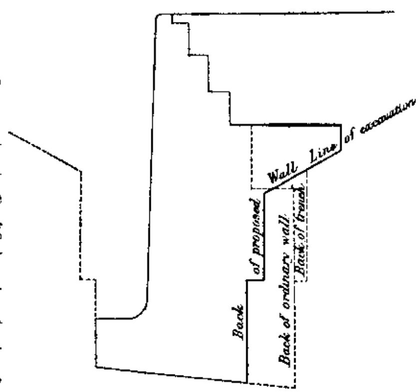
Fig. 12.



Mr Hollingworth.

be small, yet for 20 years it had withstood the severe hydrostatic pressure brought upon it by the basin being (periodically) emptied in about 15 minutes. The backing was composed of chalk cut from the cliff, and the thrust on the back of the wall was comparatively small. Lengthening the toe of a dock-wall to increase its stability would usually involve heavy cost in trench work, and the same result might often be attained at considerably less cost by reducing the width of the wall at the base, where the heel served little purpose except to add weight, and corbelling or bracketing out the upper part of the wall to obtain the benefit of the weight of as much filling as possible (*Fig. 13*). The proposed type of section gave the advantages of the old-fashioned counterfort, which increased stability at little cost but was unsuitable for deep trench work; it also followed the usual shape of the excavation and provided a better foundation for cranes.

Fig. 13.



Mr. Hope.

Mr. ALARIC HOPE thought the diagrams prepared by Mr. Wentworth-Sheilds showed in a striking manner the high pressures which had to be provided for when building walls in soft clay, and, by inference, the low unit pressures which such material would sustain as a foundation. It had long been a matter of surprise to him that so little attention had been paid to the determination of the conjugate pressures exerted by moist and saturated soils, information as to which was vastly more important to engineers than knowledge of those due to dry sand or gravel. It appeared to him that, in order to ascertain the pressures exerted by saturated sand, it was necessary to add to that due to the water the pressure exerted by the sand, remembering that the weight of the sand was reduced to the extent of the displaced water, and that the angle of repose was less than that of the dry material. He was not in accord with Mr. Wentworth-Sheilds's suggestion that the pressure of the sand in such a case could be neglected, nor did he see why, when estimating the pressures in front of the wall, that of the water should be assumed to be non-existent. Dealing with the question of the upward pressure of water under the base of a wall, he considered that

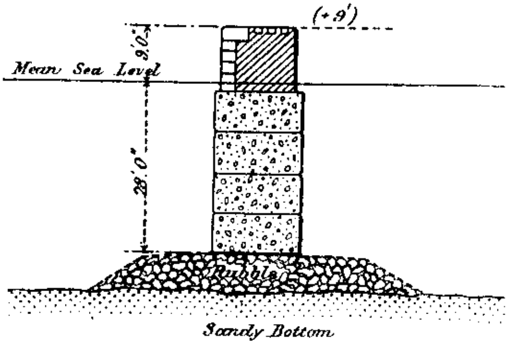
Mr. Hope. to calculate it as due to the minimum water-level would, in many cases, give too low a result. He thought that there must be many impervious, or nearly impervious, walls behind which the saturation-level stood at or about mean dock-level, and under which there was a pervious foundation. If, in such a case, the water-level of the dock were quickly lowered, the hydrostatic pressure under the wall would vary from a maximum value at the back to a minimum value at the toe, and the maximum pressure might be that due to the saturation-level. He had known more than one case in Liverpool where the concrete sill or platform of a dock-passage had fractured, during the exclusion of water for repairs, owing to the hydrostatic pressure of water which had percolated through the sandstone on which the concrete had been deposited. By drilling vertical holes through the concrete and thus permitting the free escape of the percolated water, further trouble had been avoided. He was glad to see that Mr. Wentworth-Sheilds had emphasized the importance of depth below the surface in the consideration of the bearing powers of soils. That factor was too often neglected. In considering devices for stabilizing walls Mr. Wentworth-Sheilds made no mention of weep-holes, which were desirable where the water-level in front of a wall was liable to a large range of variation, nor did he touch upon the advantages of introducing a light and stable filling behind the wall where the natural material was bad. An objection to weep-holes lay in the tendency, greater or less according to the nature of the backing, to draw material through the wall and cause subsidence behind the coping. That might prove dangerous if the quay were covered with pavement strong enough to support itself until a large cavity had formed beneath it. Such loss of material, however, might be avoided by care in designing the drains leading to the weep-holes, and by the provision of an effective valve in the face of the wall to prevent the passage of water from front to back. Mr. Wentworth-Sheilds's statement that deepening the foundations of a wall might increase its tendency to overturn could only be true in exceptional cases where the stability of the soil was very bad. In general, the resistance to overturning was increased by deepening the foundations.

Mr. Latham. Mr. ERNEST LATHAM thought that Mr. Wentworth-Sheilds's Paper was of the greatest interest, and he regretted very much that the present conditions did not allow a Research Committee on Earth-Pressures to be appointed by The Institution. He asked whether the figure of 2 cwt. per square foot for cargo super-loads had been the result of actual experience at Southampton Docks. Such a load would, of course, represent only a small portion of the stresses set up in a deep retaining-wall. On the other hand, it seemed to him that the load might not be correct. In a recent case he had designed works for a load of 8 cwt. per square foot, and had been astonished to find that the wharfingers had recently been applying a deck load of nearly 1 ton per square foot to the structure. That case was, perhaps, exceptional, as the material stored consisted of paper pulp, which gave one of the most intense deck loads that occurred. On the other hand, 2 cwt. per square foot seemed a very low figure, and if 8 cwt. were taken for superload, the lateral forces would be appreciably increased, and, where the stresses on the wall were approaching the limit of safety, the deck load might be the final stress which caused the wall to move. He had had experience of a heavy mass-concrete wall on London clay. The wall was of extraordinarily liberal proportions in the matter of width of base compared with height, but it moved forward bodily directly the first superload was applied after heavy rain. Mr. Latham.

Dr. LUIGI LUIGGI considered that the necessity for deep-water quays was felt more than ever nowadays. The fact that the Panama Canal, the Ambrose channel to New York, and the channels leading to the ports of Liverpool and Southampton, were already dredged to 40 feet depth of water, that the Suez Canal was being dredged to 43½ feet, and that the harbours of Genoa, Naples, Colombo, Singapore, Shanghai, and others were being arranged for quay-walls of 40 feet, indicated that such deep-water quay-walls would be more common in the near future. Thus the problems connected with their construction deserved all the attention of harbour-engineers. But if the difficulties of building economically quay-walls with 28 to 30 feet of water, as at present, were great, the difficulties increased in a very rapid ratio for a depth of 40 feet. Thus the two Papers, particularly that by Mr. Wentworth-Sheilds, which tended to throw light on a problem still somewhat obscure, should be very welcome. Unfortunately, the purely mathematical way of solving the problem was not sufficient, as the engineer should consider economy of construction conjointly with perfect stability. From the latter point of view the results of experience were most useful in enabling safety and economy to be combined under the various circumstances of practice and local conditions. Experience had shown that, for a wall founded under water, and having to resist the pressure of a filling completely saturated with water—whatever might be the case for an ordinary retaining-wall above water—the tendency of the wall to slide on its base was always much more to be feared than that of turning over on its toe. While he could mention many striking examples of quay-walls sliding on their foundations—and one, Dr. Luiggi.

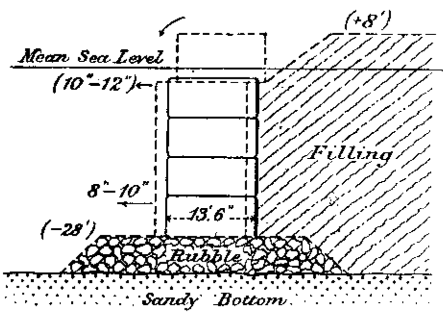
Dr. Luiggi. which had happened in Naples, was disastrous—there were not on record, at least in Italy, failures of quay-walls which had turned about their toe with any practical inconvenience. The following observations referred to quay-walls built in an almost tideless sea, that was, where low water might be 10 or 12 inches below

Fig. 14.



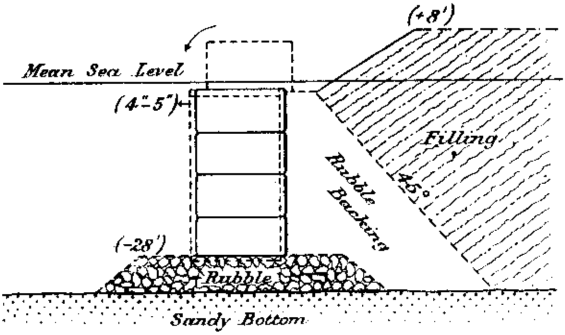
mean sea-level, and high water—owing especially to very strong, persistent winds—about 12 to 14 inches above that level. As Resident Engineer on the harbour-works of Genoa, he had had to build, in addition to the two breakwaters—one nearly a mile long, founded in 80 to 95 feet of water—some miles of quay-walls, the section of

Fig. 15.



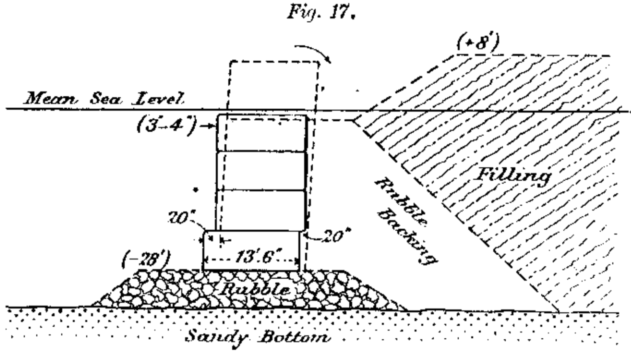
which was initially as in *Fig. 14*, but as the result of subsequent experience was slowly modified as in *Figs. 15, 16, 17*, and *18*, the last being adopted for the greater part. The quay-wall was built of concrete blocks superimposed vertically—i.e., without lateral bond, so as to leave them free to settle individually—and founded on a rubble mound resting on a bottom of sand mixed with a small proportion of mud, so that the bottom was slightly compressible. To compensate for foreseen settlement, the rubble foundation was made some 10 or 12 inches higher than indicated in the plans. When the concrete blocks were laid in place by floating derricks, an extra Dr. Luiggi.

Fig. 16.



block was laid on top of each pile, in order to accelerate the settlement of the rubble mound and of the sandy bottom underneath. The weight of the extra block, which was removed after all settlement had stopped (within 6 to 10 months), was practically equal to the weight of the masonry front wall to be built upon the concrete blocks. In that way practically all cracks in the masonry

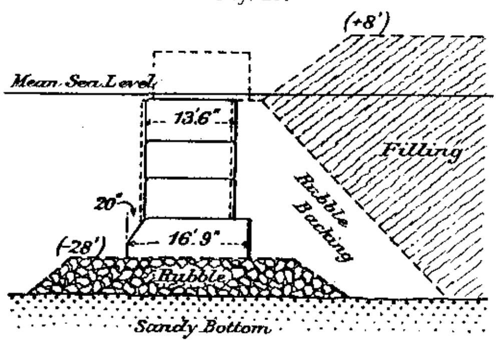
Fig. 17.



were avoided, and its front presented a very neat aspect. During the settling of the concrete blocks the filling at the back was begun, and that was always completed before the extra block was taken away. Thus all movement, vertical or horizontal, of each pile of concrete blocks was completed before building

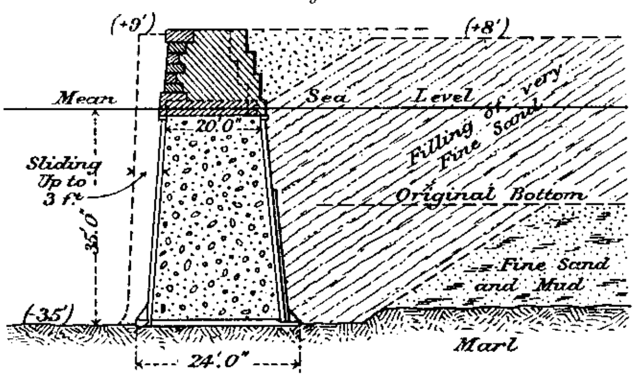
Dr. Luiggi. the masonry. The filling consisted of quarry "cleanings" (small stones, weighing less than 2 lbs., which were not permissible in the rubble), and of earth. The small stones, and also debris from buildings that were being demolished in the city, were reserved to form the backing near the concrete blocks. The filling advanced towards the wall (*Fig. 15*) and the toe of the slope pushed first against the bottom layers, and successively, as the filling increased in height, against the upper layers of blocks, till the filling was about 8 feet above mean sea-level. Under the horizontal pressure of the filling the concrete blocks began to move forward, partly, perhaps, sliding on the rubble mound, partly crushing the smaller stones of the mound; that movement was on the average 8 to 10 inches, and, in a few cases, 12 inches. The tendency to overturn was very small; only 2 to 4 inches from top to toe. As experience was gained, the

Fig. 18.



following alterations were made in the plans. Between the concrete blocks and the filling (*Fig. 16*), a backing of rubble (stones ranging from 10 lbs. up to such weight as a man could handle easily) was formed, with a natural slope of about 45°: then the filling began. The pressure of the filling acted first on the toe of the rubble backing where the latter was widest, and successively, when the pressure of the filling was less, against the thinner part of the backing. That modification of the original plan gave excellent results, as the horizontal movement of the wall practically ceased, only a few inches being observed (*Fig. 16*). The rotating movement also diminished considerably, and there only remained the vertical settlement of the structure, due partly, as already said, to crushing of the corners of the rubble foundation, and partly to the natural compression of the sandy bottom underneath. Later, to diminish the pressure on the toe of the pile, where the crushing of the rubble was more to be feared, the bottom concrete block was laid some 20 inches forward of the front line (*Fig. 17*), but then another difficulty was encountered. The wall, which before had had a tendency to slide and turn forwards, now had a tendency to overturn inwards, partly due to the weight of the rubble backing, which, compressing the sandy bottom, developed a pull downwards owing to the friction against the back of the concrete blocks, thus helping their inward movement. All trouble ceased when a wider block was adopted for the bottom layer, as in *Fig. 18*, which design gave always excellent results. The whole structure settled some 4 to 6 inches owing to the compressible soil and slight crushing of the sharper points of the stones of the rubble mound, but there had been no more sliding or overturning worth speaking of. Theoretical calculations and practical experience had shown that that was Dr. Luiggi.

Fig. 19.



the most convenient and economical section of quay-wall to be adopted under the local conditions of Italian ports. Its cost, before the war, was about 1,500 lire per linear metre; practically £20 per linear foot, at the normal rate of exchange. Now, with the general increase in prices and fluctuation of exchange, no such comparison could be made.

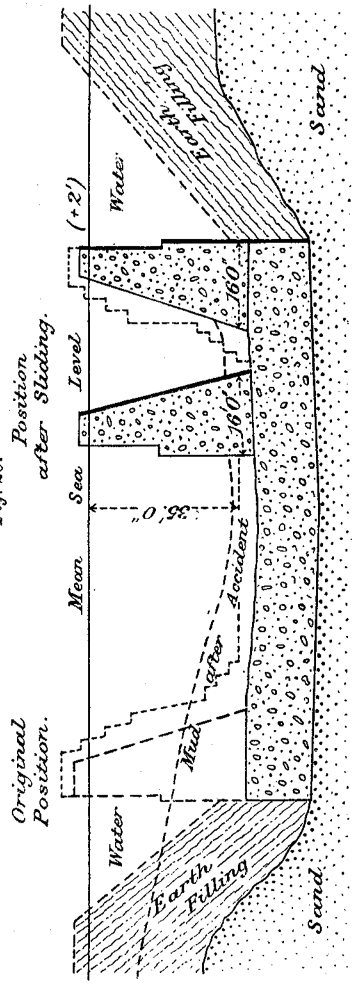
Another, and more interesting, example of sliding of quay-walls happened in Venice at the new "*Calata di Ponente.*" The quay-wall (*Fig. 19*) was formed by a substructure of reinforced-concrete caissons, nearly 35 feet deep, filled with pozzolana concrete, rather lighter than cement concrete, and by a superstructure of masonry about 9 feet above water. The caissons had flat bottoms, and rested on a layer of marl. The backing was formed mainly with fine sand from the dredgers and rubbish from demolished buildings, the latter

Dr. Luiggi. being especially reserved for the back of the caissons. The wall resisted quite well till all the work was nearly finished, but when locomotives began to run on the newly formed surface behind the wall, some movement forward was noticed, increasing week by week, especially in the centre of the wall. This movement was evidently due to the filling causing the wall to slide on the marl. It was stopped by taking away rapidly some filling at the back, driving reinforced-concrete piles in front and against the toe of the caissons, and finishing the surface with rubble stone, which caused much less pressure.

A third example—the most remarkable, and perhaps unique in the history of harbour engineering—happened to the graving-dock of Naples, where a whole side of the dock slid inwards under the pressure of the outside water and of the newly-formed filling. The graving-dock had been built in the open harbour, under water, by means of floating compressed-air caissons. That was a method of construction very much in vogue in Italy, which he had applied for the first time in Genoa, to the two graving-docks built in 1887–92, and which later on was applied to the graving-docks of Palermo and Naples, and recently in Venice. All the concrete shell of the Naples graving-dock was finished in 1908, the water was pumped out, very little percolation being noticed, just as had been the case with the three graving-docks of Genoa and Palermo, and thus the workmen began the revetment with ashlar and granite, without any misgiving. At the same time the filling outside the graving-dock was proceeding regularly by tipping the earth into the water, and the toe of the filling was approaching and rising little by little against the eastern side wall of the graving-dock. All went well for some time, and no sign of movement was noticed. However, one night the eastern side gave way quite suddenly, without any warning, and was pushed bodily forward, sliding along the invert of the dock till it came to rest against its inner western side. The enormous block of concrete put in motion, about 400 feet long, 30 feet high and 15 feet wide at the bottom, broke in several pieces, but each piece moved remained almost vertical. Happily, the accident took place at night, when no one was at work inside the graving-dock, otherwise a terrible loss of life would have taken place. The sketch (*Fig. 20*) gave an idea of the relative positions of the side wall before and after sliding. The cause of the accident must have been that the pressure of water outside, with that of some of the earth filling, began to cause a small fissure between the invert and the side wall founded on it; pressure was then developed, which had a tendency to augment the fissure and to detach completely the side wall from the invert. Then the water and mud, passing underneath the side wall, lifted it and moved it bodily forward, causing the wall to slide along the invert, till it stopped almost vertically against the other side. This remarkable example of a wall sliding on its foundations strengthened his opinion that the sliding of quay-walls was more to be feared than overturning. Dr. Luiggi.

Having given these preliminary items of personal experience, he wished to state his concurrence with all the remarks made by Mr. Wentworth-Sheilds about the forces to be taken into account when calculating the stability of a quay-wall. He considered that the formula for calculating the horizontal pressure of the filling was quite safe, provided some discrimination was made about the value of these factors, namely, w, the weight per unit of backing, and ϕ, its angle of repose. From experiments made by him on the usual materials for filling — earth, sand, gravel, debris of demolished masonry, debris of quarries, rubble, etc. —their angle of repose might range from 45° to 35° when out of water; but, when they were deposited under water, and had become completely saturated, their angle of repose diminished, and earth, especially if containing much clay, might rest only at an angle of 20°. In order to cause the least possible pressure against the back of the quay-wall, it was necessary—at least, just near the wall—to employ a material like, for instance, rubble, with the largest possible angle of repose (45°), and at the same time of such a nature that it

Fig. 20.



Dr. Luiggi. lost weight greatly when immersed in water. Rubble of calcareous nature, as obtained in Genoa, weighed 99 lbs. per cubic foot in air, and 62 lbs. when immersed in water. His experience with calcareous stone of a slaty nature, as in Genoa, and of a crystalline nature, as in Leghorn, and with their sands or crushed ballast, had given the following results :—

Materials.	Angle of Repose ϕ		Weight w	
	In Air.	Immersed in Water.	In Air and Dry.	Immersed and after some Months in Water.
			Lbs. per Cubic Foot.	Lbs. per Cubic Foot.
1. Earth mixed with sand, rubbish of demolitions, small stones less than 2 lbs.	30°–35°	20°–27°	89·5	52
2. Very fine sand . .	33°–38°	25°–28°	83·5	58
3. Sand of various sizes up to $\frac{1}{8}$ inch . .	35°–40°	30°–33°	105·0	60
4. Gravel, broken stone in pieces up to 3 inches . .	40°–45°	40°	104·0	60
5. Broken bricks, volcanic tufa, lava, masonry . . .	45°	45°	96·0	59
6. Rubble stone, mixed large and small down to 3 inches . . .	45°	45°	105·0	64
7. Large rubble, with stones from 10 lbs. to 60 lbs. ("one-man stones") . .	45°	45°	99·0	62

By using Rankine's formula, as suggested by Mr. Wentworth-Sheilds, but slightly modified thus

$$P = \frac{wh^2}{2} \tan^2\left(45^\circ - \frac{\phi}{2}\right),$$

as it was adopted usually on the Continent, the influence of the angle of repose ϕ and of the weight w of the back filling was shown more clearly. The results indicated that the pressure P decreased slowly with w, but more rapidly with an increase of ϕ, demonstrating the importance of using rubble stone for the backing immediately near the wall. The resistance of the quay-wall depended mainly on its weight, thus rendering the use of Portland cement concrete blocks advisable in preference to pozzolana concrete blocks, as used for economy in Italy, where pozzolana was most abundant. In the calculations the immersed part of the wall, which lost weight in proportion to the water displaced, should always be differentiated from the part which was always above water. As to the effect of the pressure of the water in front and at the back of the quay-wall, it might be assumed that they balanced each other (except in places where the range of tide was considerable), as with the type of quay-wall used in Italy, the water passed easily through the rubble, or between the open joints of the concrete blocks, from the front to the back. Where the wall rested on a rubble foundation the safe pressure at the toe should not be more than 3 to 3½ tons per square foot, as experience showed that, with a heavier pressure, the wall had a tendency to overturn on its toe by crushing the rubble foundation. When such results of experience were kept in view, Mr. Wentworth-Sheilds's conclusions could be accepted as combining safety with economy. Dr. Luiggi.

Mr. J. B. L. Meek remarked that Mr. Wentworth-Sheilds's Paper appeared to deal with the forces to be sustained by a quay-wall when the construction of the wall was complete and the quay was in use. There might, however, be cases where a quay-wall could be constructed in the dry, either in a trench, leaving the necessary excavation in the front of the wall to be done after the wall was built, or in the open, after the excavation forming the berth had been completed. In either circumstance the wall might have to act for some considerable time as an ordinary retaining-wall, without the assistance of water-pressure in front of it. A wall might be so designed that it was perfectly stable under the forces exerted by the water-pressure in front of it, the pressure of the earth at the back, and its own weight, assuming that water did not get under the wall and that the backing was dry. Even should water get under the wall and rise to the same level at the back of the wall as at the front, thereby causing part of the wall and the filling behind it to be water-borne, the stability of the wall might not be endangered; but should the wall be called upon to act as a retaining-wall with no water-pressure in front of it, the stability might be seriously endangered. Thus a wall that was quite stable when called upon to carry out the duties for which it was designed might have to pass through a period of dangerous instability, unless the conditions during construction were taken into consideration and allowed for in the design. Mr. Meek.

With regard to Mr. Latham's remarks about driving scarfed timber piles, Mr Meek would be glad to know whether the steel

Mr. Meek. plates were countersunk so as to be flush with the sides of the piles, or whether the plates projected their thickness of 1 inch beyond the sides of the piles; also the nature of the ground in which these scarfed piles had been driven.

Mr. Porter. Mr. E. W. PORTER, being much interested in the subject opportunely raised by Mr. Wentworth-Sheilds, wished to support his suggestion that The Institution should collect data. It appeared very desirable that members should be invited to supply information, on lines formulated by a committee, relative to the design of quay-walls under their charge, and to state the conditions under which they had been constructed, with details of any failures that had occurred, and of the remedies adopted. There could be no doubt that the absence of a general agreement among engineers, with the important desirability of safety always in view, was responsible in some cases for undue capital expenditure, as a result of the unsatisfactory nature of current solutions of the problems involved, which induced them first to prepare calculations, and then to trust to experience. He observed that the Author had not mentioned, under "Devices for Stabilizing Walls," bearing-, or keying-piles, and sheet-piling at the toe, as a preventative against sliding. It would be interesting to hear whether the Author had met with examples of walls built on, or into, bearing-piles of timber or reinforced concrete which had failed in one way or another. Another means, which no doubt the Author had tried, for lessening the lateral pressure of backing, was the removal of backing and the substitution of selected material with a steeper angle of repose. Examples and results of such a device might raise points of interest. While, generally speaking, the various forces and resistances were similar for a gravity section and for one of the type such as reinforced concrete lent itself to, it was conceivable that in the latter an item for consideration among the horizontal inward forces would be the pressure of a ship in an on-shore wind against the quay. The present-day necessity for vertical quay-faces, the consequent severer effect of propeller-scour, and any subsequent disturbance of the compactness of the ground at the toe of a wall by dredging, all affected materially the items D1, D2, and D3, enumerated in the Paper. Quay-walls under Mr. Porter's charge had suffered by the scouring effect of vessels' propellers. Data collected by The Institution would no doubt comprise profitable details bearing on that question and also on the effect of vessels of very square mid-ship section taking the ground in close proximity to the toe of a quay-wall.

Mr. A. H. ROBERTS observed that, in addition to those uncertainties of the factors governing the stability of a wall which related to the quality of the clay or other material upon which the wall was founded, there was another matter to which reference should be made, namely, the permanence of the quality of the foundation. A wall might be founded upon very hard clay, but in the course of years the clay, if it was covered with water, as in the case of a dock foundation, might become so penetrated by the water as to be softened to a degree which quite altered its value from the point of view of stability. He knew of one dock, built about 60 years ago, where the foundation had been carried very little below the bottom of the dock, since it was in very hard, stony clay. The bottom of the dock had now become softened to a material degree, and Mr. Wentworth-Sheilds's factor B2 (lateral resistance of earth in front of the toe) had become quite valueless. As a consequence, when it became necessary recently to erect travelling cranes on the quay, it was considered essential to pile under the back rail of the crane, as the wall had already, in past years, required to be tied back, and no further tax could be put upon its stability. There was no doubt that when the water of a dock stood for long periods it tended to alter the quality of the bottom, and every dock-wall should be carried deep enough to secure a permanently hard foundation. Mr. Roberts.

Mr. A. SCOTT thought that the two Papers brought forward a very old question, and both showed how little real advance had been made, in recent years, in the improvement of designs for deep-water quays. Probably in no branch of civil engineering had there been less progress during the last 30 or 40 years than in the design and construction of maritime works generally, including deep-water quays. He considered that to be true, broadly speaking, although of course many variations in details had been introduced, chiefly owing to the use of reinforced concrete. The difficulties were inherent in all deep-water works, but if the Papers should lead to renewed interest and further detailed research in connection with the subject, they would have served a useful purpose. Referring to condition A1 in Mr. Wentworth-Sheilds's Paper (the lateral pressure of the backing and of the water behind the wall), if the filling or backing, whether it was sand or rubble, or other suitable material, was permanently water-borne, with the water say at the same level (tidal or otherwise) at the back of the wall as at the front (assuming, of course, an impervious watertight wall), then it would be fair to assume the pressure to be that due to the water, plus the water-borne weight of the backing, minus a percentage Mr. Scott.

Mr. Scott. for voids. The conditions and problems met with in actual practice were so varied that hardly any two cases would be exactly alike, and it was absolutely necessary to treat each individual case independently. The collation of trustworthy data, especially from experiments on a large scale, would be useful and interesting, but it would be difficult to lay down general and reliable rules on the subject, even if such were desirable. Even if the results of calculations appeared in some cases to be unsatisfactory, it was most important that young engineers should be trained to make a careful analysis and solution, by calculations and diagrams, of every case, taking into account the exact conditions and forces relating thereto so far as they might be known. Afterwards, if the engineer, with the complete investigations before him, decided to make allowances on the side of safety, well and good.

Mr. Latham's Paper might more correctly have been entitled "Deep-water Piers and Jetties," or "Deep-water Wharves." The terminology used was somewhat equivocal, and rather loosely employed. The words jetty and quay were used for the same structure. It was very desirable that the nomenclature or terminology used should be exact and that it should describe adequately the structure under discussion, so as to convey a definite idea of its construction. The term "quay" was generally applied to a solid, continuous, marginal structure, more or less parallel to the sea-shore, or to the banks of a river, or to the sides of basins or docks, with a solid backing or filling behind it. Such a quay might also be termed a marginal wharf—using the word "wharf" as a general term for any berth, in any position, at which vessels or cargoes might be loaded or unloaded. A pier or jetty might be used as a wharf, and generally was so used; but neither a pier nor a jetty should, strictly speaking, be termed a quay. As to what depth should constitute an up-to-date deep-water quay, it was rather difficult to suggest a limit, but when designed to provide accommodation for large modern ocean-going cargo-vessels, looking to the necessity for economy in handling and despatch, the depth alongside the quay should not be placed at less than 30 feet at low water, whatever the rise of tide might be. The Thames was particularly deficient in that respect; probably there were not more than one or two berths (not quays) above Tilbury with a depth of 30 feet at low water, apart from the docks. He was of opinion that in a first-class port it should, if possible, be laid down as a general rule that large ocean-going vessels should be able, fully loaded, to go alongside the quays at any state of the tide. They should never take the ground. It did not seem practicable, or desirable, to have any settled policy on design, because the class of construction which would be adopted in any particular case might depend upon many things—requirements, class of freight to be handled, the traffic and the equipment and loads to be carried, situation and nature of the ground, and total first cost, etc. Open piled work was seldom used for permanent quay-works, but for piers, jetties, or wharves it was eminently suitable. Timber had such a short life at most places abroad that it could not be economically used even for piers. For purely commercial undertakings it might not be economical or advisable to install works which would have a longer life than 25 or 30 years. In considering the undermentioned different types of construction for piers or jetties, their economic advantages as regarded speed of construction and cheapness in total first cost might be placed roughly in the following order:— Mr. Scott.

(1) Timber piled work.
(2) Reinforced-concrete piled work.
(3) Cylinder construction.
(4) Walls of various types with solid filling behind, including ordinary mass-concrete and masonry walls; concrete block walls; walls with substructures of concrete caissons; and walls constructed of reinforced-concrete trestle and slab work, with solid backing.

This classification would probably have to be varied beyond a certain limit of width in the pier. For quays, and for very wide piers, walls and solid filling were generally the most satisfactory class of construction, and for a moderately long life they were the cheaper in the end.

Mr. L. J. Speight remarked that, in the hope that observations by one intimately concerned in the construction of docks and deep-water quays might prove to be not without interest, he ventured to take part in this discussion. Mr. Latham referred to two forms of reinforced-concrete construction, and expressed the view that there appeared to be no settled policy on questions of design. If it could be demonstrated beyond question that with reasonable precautions reinforced-concrete structures could be constructed in such a way that sea-water could not percolate through the concrete to the steel, the doubts of many engineers as to the durability of reinforced concrete would be removed. That consideration might in some measure account for the apparent lack of agreement on questions of design referred to by Mr. Latham, in that some engineers thought it desirable to take additional precautions by way of Mr. Speight.

Mr. Speight. surrounding piles below L.W.O.S.T., that was to say where subsequent close observation was impossible, to meet that contingency. Quite recently Mr. Speight had come across a case where the reinforcement in concrete piles was provided with 2 inches of cover—the explanation given by the designer being that it was necessary to have additional cover in marine work to prevent percolation of water to the steel. Greater risk of percolation arose in those members of a structure which were pre-cast; for example, piles and under-water bracing. Unless the utmost care were taken, excessive bending moments were set up in the process of hoisting and pitching reinforced-concrete piles, causing cracks in the concrete of a more or less serious nature. If, added to that, there were severe driving through hard strata where a given depth of penetration, as opposed to "set," of the pile was required, it would be seen that there was justification for doubt as to whether water would not percolate to the steel, the tendency to percolation being greater as the depth of water increased. The increase beyond normal thickness of cover to steel in the case of pre-cast members would not provide greater protection of the reinforcement, but on the contrary it was harmful, in that the concrete would certainly fracture more readily in handling; and experience of driving piles with varying thicknesses of concrete cover had proved conclusively that vibration due to the driving had caused the cover to shell off when it exceeded about $\frac{7}{8}$ inch in thickness. In cases of open-piled structures, where the depth of water exceeded about 18 feet at L.W.O.S.T. some form of lateral stiffening was desirable. Pre-cast reinforced-concrete bracing fixed under water with sleeve connections was very costly and would appear to be unsatisfactory, having regard to the element of uncertainty as to the degree of success attained in fixing. The cylinder method of construction eliminated the necessity of bracing, in that self-supporting columns were substituted for open piling. The difficulty experienced in executing cylinder forms of construction occurred in the portion of the work below L.W.O.S.T. Of the two methods of cylinder construction referred to in Mr. Latham's Paper, the use of temporary cylindrical moulds was, in his opinion, likely to produce the better result, because if the moulds were subsequently stripped—as they should be—any defects in the operation of depositing the concrete under water became apparent and could be rectified. If the moulds were not stripped, or in cases where pre-cast reinforced-concrete tubes were used with the object of forming part of the cylinder construction, there were no means of ascertaining the degree of success attained in the very

Mr. Speight.

important operation of depositing the concrete core. In cases where in the design of the work the interior piles were in close proximity to the face of the cylinder, consideration should be given to the question of accurate driving of the piles; otherwise it might be found impossible to encircle them with the mould (or tubes) of the required diameter. The depth to which the cylinders should penetrate into the bed of the river should not be less than 3 feet—regard also being had to the nature of the strata, the angle of repose of the material and its tendency to "draw down" under the influence of scour, which was increased in force at the periods during which vessels were moored alongside, particularly where a vessel's hull was in close proximity to the bed of the river. In one case within his knowledge the lower ends of the cylinders were subsequently exposed by the drawing down and scouring of the slope, with the result that the cylinders in question were suspended above the ground. It was desirable that the bed of the river in the area surrounding the cylinders should be excavated and made up with concrete bag-work, to prevent any possible leakage of concrete from the inside of the cylinder during the operation of concreting, care being taken that no obstructions were left on the bed of the river likely to cause damage to vessels lying alongside the jetty. Seeing that the stability of that type of structure depended largely upon having solid concrete cylinders, precautions taken to attain that end should be of a positive nature. His experience was that the highest degree of success was attained when the process of depositing the core was carried out uninterruptedly from the start to a minimum height above L.W.O.S.T. The concrete should be deposited through a light steel water-tight tube fitted with a water-tight foot-valve operated from above, the top of the tube being fitted with a hopper into which the concrete could be fed rapidly. The tube was of such length that when the foot-valve rested on the bottom of the cylinder concrete could be fed into the hopper above the level of the water. The number of tubes to be used was governed by the size of the cylinder, the quantity of concrete to be placed, and the time available between tides. Suitable arrangements were made for rapid lifting and lowering of the tube, which, at the commencement, was raised so that the foot-valve was above the level of the water. The foot-valve was opened to release any water from the tube and then closed, after which the tube was filled with rich semi-dry concrete. The tube, with hopper attached, was next lowered into the cylinder and allowed to rest on the bottom, after which the valve was opened. The tube was then gently lifted a few inches and the concrete flowed away

Mr. Speight. from the foot. More material was then fed into the hopper at the top. The greatest care was necessary in the manipulation of the tube to prevent the loss of the concrete "priming." The top of the concrete in the tube should be visible throughout. If for any reason the charge in the tube were lost, the tube should be immediately withdrawn and recharged with the foot-valve above water-level. The deposited concrete should be disturbed as little as possible, and ramming or tamping should not be resorted to. Where circumstances had necessitated the operations being suspended before reaching L.W.O.S.T. it had generally been found on subsequent examination that a crust had been formed varying in depth up to several inches over the area of the cylinder and consisting of a soft, greasy substance somewhat of the consistency and appearance of glazier's putty. This was probably due to cement rising from the concrete and mixing with suspended matter in the water, and then being deposited as a film over the top of the concrete. Where that occurred the surface had to be thoroughly cleaned before recommencing concreting. It would be seen that the operations were comparatively costly, but where care was taken, success would be achieved and demonstrated on the removal of the temporary steel cylindrical moulds (if used), when the face of the cylinder would present to the touch a smooth, hard surface. The cost of maintenance of reinforced-concrete works arising out of ordinary usage was negligible in comparison with timber and iron structures, whilst damage caused by collision was confined to a much smaller area, so that, although the unit cost of reinforced concrete was higher than that of timber or structural ironwork, the total cost of repairs was likely to be less.

The system of fendering, which was a point of importance, did not always receive the consideration it merited. In the case of a large ship with horizontal and vertical overlapping seams to the hull-plates, considerable damage was caused to fendering by the lipping of the projecting-plates where the vessel was bearing against the structure whilst being moved, or when lifting and falling with the tide. The most common practice was to provide vertical fenders only, spaced at comparatively wide intervals. Where fendering was so designed he had found that large ships caused excessive wear and tear, resulting in the destruction not only of fenders, but also, particularly in the case of timber structures, of the piles to which they were attached. Fendering should be arranged with both vertical and horizontal members, and in such a way that the main members were horizontal and not more than

4 feet apart vertically, with convex rubbing-faces projecting about 2 inches in front of the verticals, which reinforced the bolts of the horizontal members in resisting the tendency of a vessel to wrench off the horizontal fenders by its vertical movements. In such an arrangement lateral movement of a vessel bearing against the structure could be effected without lipping, as clearance was provided for the vertical seam of the plates in a vessel's hull, whilst the edges of the convex horizontal fenders, being clear of the line of projecting seams, would allow the vessel to rise and fall freely with the tide. Owing to its tough fibrous nature, Canadian rock elm, although costly, was probably the most economical timber to use for that purpose. The best results were obtained when the fenders were attached directly to the main structure. Intermediate packing between the main structure and the fenders should be avoided altogether if possible, as it entailed the use of longer bolts, which, acting as levers, accentuated such strains as were set up in the fenders. With regard to the method of mooring vessels illustrated in *Figs. 3* of Mr. Latham's Paper, he ventured to express the opinion that the difficulties arising from tidal variation could be considerably lessened by arranging longer "springs". In the instance illustrated the springs might be led from the starboard bow and starboard quarter to the stern and stem dolphin bollards respectively. He had noted at several wharves that such a method was most usually adopted. Mr. Speight.

Sir Francis Spring.

Sir FRANCIS SPRING had found Mr. Latham's Paper somewhat disappointing, for although entitled "Deep-Water Quays, General Considerations of Design," a perusal of it showed it to be confined to considerations of the design of a limited character of jetty for special cargoes. Both designs dealt with in the Paper appeared to contemplate a class of jetty for use with cargoes capable of being pumped ashore, such as oil and so on; for the deck of the Thames Haven jetty was only 13 feet wide (except at the two dolphin ends), leaving no room either for cargo that must be railed or that must be stored, except in so far as it might be practicable for such cargo to be carried along a narrow gangway for a distance of some 100 yards from hatches to shore. Indeed, in the case of the second jetty dealt with, there would appear to be no provision for bringing any given hatch up to the narrow, accessible end of the jetty. Assuming, however, that the Paper was intended to be of more general application than the limited one indicated above, Mr. Latham's remarks as to the most economical design of structure and as to berthing facilities might be read with interest. Reasons were offered for a reinforced-concrete system of piling so substantial

Sir Francis Spring.

that the structure should not suffer over-much from the consequences of bumping during berthing operations. Undoubtedly, the inevitable bumping sooner or later caused cracks—perhaps at first only hair-cracks—which, at least in salt water, could not but lead, sooner or later, to the corrosion, and the consequent swelling, of the steel reinforcement. Because of the occurrence of such cracks during the process of mooring, or due sometimes perhaps to "range," Sir Francis Spring had found it necessary, in tropical waters, to eschew the use, at least for fenders, of 14-inch reinforced-concrete piles having $1\frac{1}{2}$ to 2 inches of fine concrete overlying the steel reinforcement, and to substitute long timber, with all its disadvantages in the form of susceptibility to attack by teredo and other pests, the evil results of which were so well illustrated by the specimens in the museum in the Institution premises—a most instructive exhibit. In Sir Francis Spring's opinion the maximum advantage would appear to be offered by a system of steel cylinders surrounding the pre-driven pile or piles, the space between being filled with fine and strong concrete, sent down in tubes, or "tremies." The containing cylinders would at least last for a few years; but even if they were designed for removal after the filling should have set, it ought to take a good deal of bumping before cracks could penetrate so far into the comparatively substantial concrete filling as to corrode the reinforcing bars in the contained pile. As pointed out by Mr. Latham, an alternative to the scheme of sheathing was to slip short reinforced-concrete cylinders over the piles and to fill the space between with strong concrete. With that system the reinforcement in the outer cylinders was certain to get corroded through inevitable cracks, but the interlying concrete would be found substantial enough to prevent—at least for quite a long time—access of salt water to the reinforcement in the interior pile or piles. In neither case, whether permanent or temporary steel cylindrica sheaths were used, or permanent cylinders of reinforced concrete, would it be wise to make the individual cylinders too high, because it would be desirable, in order to prevent hollows in the concrete filling, for divers to ram the concrete, as it fell from the tube, so that all interstices should be thoroughly filled. His experience of the difficulties attendant on the rises and falls of tides, and their effect on shore and breast-ropes and springs, had been confined to tidal effects in seas offering only a 3- to 5-foot tide. It was easy to see, however, that attention to such matters, when tides were four or five times those heights, was of considerable importance, and that it must be difficult to ensure such care, at all hours of the day and night, as should preclude considerable damage to gear which, especially nowadays, cost a good deal of money. It remained to be seen whether engineering and nautical ingenuity might not yet avail for the design of a satisfactory mooring system that might be trusted to look after itself, regardless of tides. With respect to the scarfing of long timber piles, he thought he had mentioned already, in another Paper, that his own practice was to butt-end—not to scarf—such piles, fishing them by means of 15-foot lengths of double-headed rail, laid flat along all four sides and bolted through the webs with two or three bolts above and the same below the butt joint. Piles so jointed could be driven without risk of shivering at the joint. He had jointed long, heavy, reinforced-concrete piles in a similar manner, the bolt-holes having previously been left when the piles were being cast. At the end of his Paper, Mr. Latham referred to the serious damage likely to be caused to ships' bottoms, and to the periodic soundings necessary, if the sea-bed on which a vessel rested—if it had so to rest—should prove not to be level. The only method of sounding which Sir Francis Spring had found to be reliable was to sling a 30- or 40-foot rail from a crane, horizontally, and at right angles to the quay-face, and to cause the crane to run along the quay, and then, by a bucket dredger, and, if necessary, by divers, to remove any irregularity thereby revealed. It suggested itself that two opportunities were open, in connection with the subject of Mr. Latham's Paper, for inventors, in co-operation with harbour authorities and experienced mariners, to devise, (*a*) some method for keeping a vessel close up against the fenders of a quay, which should not need special attention because of tides, and (*b*) a satisfactory method, and one not over-wasteful of time, for securing a uniform bottom alongside a quay in cases where vessels must rest on the ground, partly water-borne. Sir Francis Spring.

In Mr. Wentworth-Sheilds's Paper there was no reference to the stability conditions of walls founded on sunken monoliths: that had been dealt with in Sir Francis Spring's and Mr. H. H. G. Mitchell's Paper.[1] The four-berth continuous quay in question, with foundations founded 13 feet deeper than the deepest contemplated sea-bottom, cost £49,000 per berth, or £66 per linear foot. As would be seen on reference to his reply [2] to the criticisms on the Paper in question, and to the diagram illustrating it, the whole face of that quay moved forward when the ground was dredged from in front and the filling was deposited at the back of it. The forward movement ranged from about 4 inches to 6½ inches—a movement invisible to

[1] Minutes of Proceedings Inst. C.E., vol. ccvi, p. 2.
[2] *Ibid.*, p. 64.

Sir Francis Spring.

the unaided eye in a length of some 3,000 feet of structure, but alarming enough when it first declared itself, because it did not seem possible to be sure where, or if, it would stop. The conclusions to which he came, after some years of observation of that quay, was that the movement was largely due to the disturbed and unstable condition of the strata penetrated, as the result of the sinking of the wells or monoliths by grabbing, and that as soon as the soil in front of the structure had solidified sufficiently by lapse of time and by the penetration of its interstices by fine mud and silt, there need have been no fear of further movement. He believed, therefore, that the entire quay would have remained stable, refraining after the first 4 or 5 inches from further movement—4 or 5 inches easily offset by an equivalent lean-back prior to dredging or filling—if, instead of resort being had to 28 feet thickness, all the wells had been made only 24 feet thick, as seventeen of them were in fact made. A lean-forward of 6 inches, in the height, 84 feet, of the structure in question, would be caused by a sinking of less than 2 inches of the forward edge at the toe of a well 24 feet thick. As experience was gained, this was offset, as stated, by leaning the monoliths a bit back and sinking them with a backward inclination, thus allowing them to come forward nearly straight when the dredging in front and the filling in rear had been carried out. The pressure due to water percolating to the back of a continuous quay ought never to be much more than that due to the difference in height between tide level in front and the momentary level of the impounded water in the rear—should the backing material be such as to allow any impounding. That back-water pressure might, presumably, be got rid of in whole or in part by means of suitable weep-holes. Although the Paper purported to concern itself only with deep-water quay-walls, it brought to Sir Francis Spring's memory many cases, which he had had to investigate, of retaining-walls in railway-cuttings that had failed owing to water getting access to, and doubtless flattening the angle of repose of, surcharged earth backings. Even worse than that, he had known cases where gypsum-charged earth had become water-soaked, resulting in, apparently, irresistible overturning pressure. In his comments on Mr. P. M. Crosthwaite's and Mr. A. R. Fulton's Papers[1] he had referred to these railway retaining-walls, and also to certain experiments carried out a good many years ago in Ireland, he believed by Mr. R. Mallet. He had now found, however, that the experiments in question were made not by Mr. Mallet

[1] Minutes of Proceedings Inst. C.E., vol. ccix, p. 319.

Sir Francis Spring.

but by Mr. Jacob Owen, who described them in vol. i of the Transactions, for the year 1844, of the Institution of Civil Engineers of Ireland. Not much information, however, was derivable from those experiments. Practical experiments on a scale likely to give useful and reliable results were expensive things to carry out, and, on the other hand, laboratory experiments could not be considered as wholly reliable. He therefore ventured to put forward the suggestion that engineers in executive charge of heavy practical work might often find opportunities, as he had done, for making their work contribute to knowledge of engineering science, as it might very fairly do in many cases, by being put to the expense of the carrying out of experiments on a full-sized practical scale, in aid of the evolution of reliable formulas whereby, in the future, similar work might be carried out with greater assurance and with less reliance on the "factor of ignorance."

Mr. Walmisley.

Mr. A. T. WALMISLEY was glad to find that Mr. Latham favoured the travel of tidal water in the River Thames, which river was remarkable for its constancy of flow, it being generally conceded that the higher tidal water could be brought up a river the better. As to his comparison of open-piled structures and concrete piles or concrete-cylinder construction, any exposed concrete in marine work needed the protection of timber fenders, whether the concrete was reinforced or not. The approach jetty to the detached landing-stage shown in *Fig. 1* commended itself, because when an open-framed landing-stage was attached in front of a solid pier or steep bank, a rebound at high water might occur off the wall, the effects of which would be reduced by the cushion of water between the wall and the face of the stage, thus minimizing the tendency to strain the off ropes of a vessel berthed at such a landing.[1] Owing to the exposed position of a landing-stage open to the sea, the constant tidal action caused soft wood to chafe and wear loose at the fastenings, thus weakening the stability of the structure. Referring to Mr. Latham's remarks upon Oregon pine for piles, some notes thereon and also upon the superiority of hardwood, used by Mr. Walmisley at Dover, for exposed pile work, would be found in the discussion of Mr. I. C. Barling's Paper.[2] While the method adopted by Mr. Latham for lengthening piles avoided the tendency for a pile to split, by using a butt joint in the timber, the connection by means of four plates, one upon each side, would appear to weaken the pile

[1] Minutes of Proceedings Inst. C.E., vol. ccix, p. 243.
[2] "The Reconstruction of the Tyne North Pier," *Ibid.*, vol. clxxx, p. 210.

Mr. Walmisley. considerably, due to the numerous bolt-holes required. Any tendency to split a pile usually started where the timber had been pierced, and hence it was desirable to keep the section as whole as possible. Mr. Walmisley therefore recommended a scarf, in blue gum piles, 18 inches by 18 inches, when main piles over a stock length were needed. Such a case occurred with the deep-water landing-stage at Dover.[1] The scarf was 4 feet long, and the two scarfing-plates were 8 feet 10 inches long by 1 foot 6 inches wide by $\frac{5}{8}$ inch thick, held by eighteen $1\frac{1}{2}$-inch bolts. Owing to the difficulty of bracing long piles below L.W.O.S.T. Messrs. Baker and Hurtzig, who acted as consulting engineers for the pier-widening contract executed at Dover by Messrs. S. Pearson and Son, recommended lower walings in the above-mentioned landing-stage just above the level of low water, as well as driving the piles into the bed of the foundation, in order to secure as much stability as possible. Tasmanian blue-gum piles were said to be obtainable 82 feet long by 20 inches square, and that timber was adopted on account of its weight (75 lbs. per cubic foot), exceeding that of water, so that, in the event of accident, they would not float as wreckage, but would sink, relieving the contractor of further risk of damage. The scarfing-plates were placed parallel to the face of the stage, so that the connecting bolts were transverse, while the waling and deck beams thereto maintained the stability of the structure longitudinally. The scarf which was adopted in the case of the blue-gum piles was near the shoe of the pile, and partly embedded in solid ground. The pile-shoe was of cast iron, with chilled points 2 inches up, 8 inches square on top, and with wrought-iron straps cast in the shoe, having countersunk holes above the shoe for connecting to the pile. A splice with two plates, $\frac{5}{8}$ inch thick, connected by eighteen galvanized bolts, $1\frac{1}{2}$ inch in diameter, might be permissible for uniting an upper length of pitch-pine pile to a lower length of blue gum pile. The main piles at Dover were driven through water and mud into chalk to a minimum of 8 feet, as the depth of pile unbraced below low-water level demanded security of the toe of the pile. In some cases a minimum of 10 or 12 feet into the chalk might be required, so as to obtain a firm hold for the foot of the pile. The deep-water landing-stage at Dover was 792 feet long and 20 feet wide, the main piles being placed 11 feet 3 inches apart between centres, with pitch-pine filling timbers, bracings, bearers, and decking, elm fenders, and, at the transverse

[1] Minutes of Proceedings Inst. C.E., vol. ccix, p. 92.

timber frames, wrought-iron tie-rods, etc. All walings, deck-beams, and cleats were checked on to, or let into, the adjoining piles to a sufficient extent to secure a fair bearing without injury to the pile, and the surfaces in contact were dressed or trussed with that result in view. The upper walings were of pitch-pine, the lower walings and portions not easily accessible for repair were of blue gum. The position of the bollard dolphins in Mr. Latham's figure was perhaps open to criticism, but depended upon location. The trouble in a stage attached to a solid pier was that there was an absolute lack of elasticity; usually this was obtained by spring or coir ropes, as wire rope had not sufficient, if any, elasticity. An isolated dolphin needed to be amply stable, and in some works a concrete-cylinder foundation had been adopted, in which the framed pile-work above water was anchored. Mr. Walmisley agreed that a heavy ram with a small fall was preferable to a lighter ram with a greater fall for pile-driving, which was to be regarded as a pushing action, and not as a hammering action. Concrete piles had been employed on some works for deep-water quays, so as to obtain a good foundation. Tests on reinforced-concrete structures had been well discussed in a Paper by the late Sir James B. Ball,[1] and Mr. Wentworth-Sheilds had previously given valuable contributions on the subject.[2] Mr. Walmisley's views on the working of a grab dredger had already been expressed.[3] Mr. Wentworth-Sheilds's diagrams, showing the outward and inward forces to be considered in the design of a quay-wall, were instructive. The late Sir Benjamin Baker investigated the bearing power of sand. After loading the upper surface of a box filled with sand until no more sinking would result, he struck the sides of the box with a sledge-hammer, when further subsidence resulted, showing that the particles of sand had rearranged themselves. The pressure of material at the back of a wall was so dependent upon its condition, whether dry or wet, compressed or loose, rammed or tipped in, that an engineer must be guided more by experience than by any of the mathematical considerations involved; but inasmuch as all contingencies should be provided for at the outset, Mr. Wentworth-Sheilds's classified conditions were usefully expressed. Mr. Walmisley.

Mr. Wentworth-Sheilds. Mr. WENTWORTH-SHEILDS confirmed Mr. Latham's experience that piles made up of two lengths of timber were effective in

[1] Minutes of Proceedings Inst. C.E., vol. cxcix, p. 123.
[2] *Ibid.*, vol. cxcv, p. 50. [3] *Ibid.*, vol. cciii, p. 276.

Mr. Wentworth-Sheilds. regard to both driving and the strength of the structure. They should be butt-jointed, and if the fish-plates were bent round at the edges so as to be of channel section, great stiffness could be secured. The chief objection to them was that the butt joints formed a point of attack for marine borers. He did not admit that reinforced-concrete structures were more liable to fracture under impact of ships than were timber structures. Generally it was the other way round, as the monolithic character of the connections in reinforced concrete gave it extraordinary stiffness. Such structures should, however, be provided with timber fenders, which should be packed off from the concrete in such a way that blows were transmitted only to points which were strengthened by bracing. He agreed with Mr. Latham that serious damage occurred to some ships if they grounded, and if the bed of the berth was not truly level, but he was doubtful whether the scientific dredging, which was recommended to remove irregularities, had yet been invented. If it had, he would be grateful for a description of the process. A fairly level bed could be made with a ladder dredger, but it was difficult to obtain one which would dredge close against the quay. A dipper dredger would probably do the work well, but it was practically unobtainable in England.

Mr. Latham. Mr. LATHAM, in reply to the Correspondence, stated that he was gratified at the general interest the subject of deep-water quay design had aroused, and he agreed that the question of nomenclature was a difficult one. In most rivers and estuaries it was nearly always necessary to advance any quay-face well into the river, in order to secure a sufficient depth of water when dealing with modern steamers of large tonnage. It was seldom that such depth of water could be secured by means of a solid structure, except in the case of a quay-face in an artificial harbour, such as Dover. If the term "jetty" had been used to describe such works as were referred to in his Paper, he thought there would have been even worse confusion of terms, as to his mind a jetty consisted of a structure projected into a river or from an open coast, ending in merely a small pier-head, and not provided with a berthing-face several hundred feet in extent. He agreed, however, that some decision as to nomenclature, and definitions of the words "quay," "jetty," "wharf," etc., were necessary. There were so many terms loosely employed by civil engineers that it would be a real advantage if the British Engineering Standards Association could be induced to take up the question of standardizing and defining the principal structural terms in common use. With regard to the remarks of Mr. A. Scott, his own experience confirmed the order of cost and rapidity of construction given in Mr. Scott's communication. Sir Francis Spring would, no doubt, recognize that the question of deep-water quays and jetties in this country was probably one which confined itself to the study of accommodating ships employed in general trades. Apart from water-borne landing-stages such as the famous Liverpool landing-stage, he was somewhat doubtful if passenger traffic and general merchandisé could ever be dealt with satisfactorily at a deep-water quay where there was any considerable rise or fall of tide, and he regarded the existing extensive works at Tilbury as a great experiment. The jointed Oregon piles referred to in his Paper were driven into Thames ballast, and the cover-plates at the joints were allowed to project beyond the face of the pile, it being considered unadvisable to recess the piles on account of possible weakness of the joint. Mr. Latham.

Mr. WENTWORTH-SHEILDS, in reply, remarked that it was evident from the Correspondence that, although opinions differed as to how to estimate the forces which made and marred the stability of a deep quay-wall, the majority of contributors were impressed with the importance of collecting information about the conditions and behaviour of various existing quay-walls, and of trying to evolve a rational theory of stability. Many of them quite rightly insisted that the designer must be guided by experience, but nearly all realized that it was not possible to apply the results of experience to their problems without analysing the various forces at work, or, in other words, without making use of some theory of stability. For instance, say that a certain wall had failed. When rebuilding it, the question arose whether it would be best to thicken it, deepen it, give it a wider toe, pile its foundation, or improve its backing? Which of the many well-known stabilizing devices should be adopted in the particular case? To answer that question it was essential to ascertain as nearly as possible where the weakness of the wall occurred. That involved the rough determination, however, of the various forces and resistances at work on the wall. Unless that analysis were made with some degree of correctness, the result might be at best an extravagant design, and at worst another failure. Or, again, a new wall might have to be built under slightly different circumstances from a former one which had stood well. The backing might be softer, or the substratum more slippery. What alteration should be made? Here, again, in order to make experience serviceable, the forces imposed on the two walls would have to be analysed, and the effect of any proposed alteration in design calculated. Mr. Went-worth-Sheilds.

Professor Luiggi's remarks illustrated how a theory of stability

Mr. Wentworth-Sheilds. had assisted him in interpreting his experience, and utilizing it for the production of an efficient quay-wall. Several writers had expressed their views on the lateral pressure exerted on a wall by a flooded backing, and most of them seemed to think that it would be greater than the pressure of water alone. Although that view was no doubt correct with an open backing like rubble stone, it was, in the case of flooded sand, probably too safe to represent actual facts. On the other hand, as others had pointed out, a very soft clay or mud might exert considerably more pressure than water; but it was probable that a soft clay did not get "flooded," as several cases had occurred where a wall with such a backing had been stabilized by admitting water in front of it. He hoped that some researches would before long be made to decide the point.

Sir Francis Spring's remarks on the resistance of earth in front of monoliths were important. Such monoliths were often sunk into soft clay which offered but little frictional resistance under the base of the wall, and their stability was largely due to the fact that they were sunk well below ground, and consequently that the "toe" resistance in front of them was considerable.

Mr. Porter asked for an instance of failure of a wall built on piles. One instance was the old outer dock-wall at Southampton, which was about 40 feet high and 12 feet wide at the base, with occasional buttresses. It was built on and backed with sandy clay, and under the toe was a line of sheet piling. It had moved forward in several places and had shown signs of overturning also. No doubt the piling had saved it from complete collapse.

Another interesting point which several correspondents had dealt with was the upward pressure at the heel of a wall due to the weight of the backing behind it. Mr. Buckton, like many others, contended that that pressure should not be ignored. Presumably they advocated that it should be considered as a definite upward vertical force on the wall in addition to those enumerated in the Paper. Mr. Beare had pointed out that, even if that were so, the centre of reaction need not necessarily fall within the middle third, as the intensity of pressure under the base need not vary uniformly from front to back. The argument appeared to be sound.

As to the effect of deepening a thin wall, referred to by Messrs. Hope and Beare, it could be shown that in certain cases the centre of reaction was thus brought nearer to the toe of a wall, and that consequently the crushing stress on the earth beneath it was increased. It was true, however, that in such cases the wall was already unstable. There was no doubt that the dock-engineer's real difficulty was to design a wall which would not move laterally, especially when built on a slippery clay, and in such cases the resistance of the earth in front of its toe was of great importance, and the effect of deepening the wall was therefore to increase its stability. Mr. Wentworth-Sheilds.