Correspondence.

Dr. Brysson Cunningham.

Dr. Brysson Cunningham remarked that the arrangement of the north quay of the northern basin undoubtedly left much to be desired, as Mr. Ash himself admitted, owing to the unfortunate impediment of the mosque. He inclined to the view that rather than have a quay consisting of two relatively short, disconnected lengths with different alignments, it would have been preferable to run a straight line from the extreme north-west corner of the basin, tangentially to the "bullnose," to a point on the eastern quay, approximately 520 feet from the north-east corner. That arrangement would appear to possess the following advantages: (*a*) it would have afforded an uninterrupted length of about 2,540 feet, only 40 or 50 feet less than the aggregate serviceable quayage of the combined north and east sides of the basin, measured to the same point on the east wall, while, as against a maximum of four berths available, as shown on the plan, it would have provided a minimum of four 600-foot berths (with an appreciable margin) and allowed for five 500-foot berths without difficulty; (*b*) it would have saved the expenditure of some £55,000 on the construction of 430 linear feet of practically useless quayage round the "bullnose"; and (*c*) it would have lent itself even better than the existing plan to the disposition of the rail-tracks serving the quays and the marshalling-yard.

As to the design of the quay-walls and the calculation of earth-pressure, it was a matter for surprise that in computing the thrust of a clayey backing, English engineers continued to rely on Rankine's formula, which had not been intended for that purpose, being specifically based on the assumption of non-cohesive, granular material. There was really no justification for its application to clay, and the only excuse to be urged for the practice was that the results so obtained were generally "on the safe side." That might be so, but there was also the possibility and even the likelihood in many cases of an excessive margin of safety leading to needless expense. French engineers had devoted closer attention to the matter, and it would have been interesting to consider the conditions at Calcutta in the light of the theoretical researches of Professor Résal, coupled with the experimental determinations by Messrs. Jacquinot and Frontard of the cohesive value of clay.[1] Professor Résal had pointed out that the free surface of argillaceous earth might stand at an angle much greater than ϕ, and might even,

[1] C. Aubry, "*Cours de murs de soutènement.*" Paris, 1925.

Dr. Brysson Cunningham.

when quite unsupported, maintain verticality for an appreciable height, dependent on the degree of cohesion, which, in turn, was a function of the moisture-content. All engineers knew of banks or retaining-walls which stood in practical defiance of theoretical considerations. The late Mr. P. M. Crosthwaite, M. Inst. C.E., had cited a striking example.[1] With regard to the adoption of well monoliths, the system of well foundations in unstable strata, indigenous to India, had spread to some European countries, notably for quay-walls in the British Isles and, in recent years, in Italy. But in Northern Europe the problem was solved on different lines. Quay-walls of considerable height were almost invariably supported on piled foundations, at appreciably less cost than the cylinder-sinking at Calcutta. He had visited, during construction, the new Griesenwärder Hafen at Hamburg, and other recent port works at Bremerhaven, Bremen, and Ymuiden, and had seen the walls there—which afford berthing accommodation not inferior to that at Calcutta and in some cases of greater depth—built on a foundation consisting of a wide low-level platform, supported on long raking piles, with or without a frontage of sheet piling. The system answered admirably, and it would certainly appear to involve considerably less expenditure.

In considering the method of dealing with the entrance works (the lock and graving-docks) in which an effort seemed to have been made to secure a watertight enclosure by means of a ring of cylinders, had the engineers taken cognizance of the system of filtration-wells employed so successfully in the construction of the Kruisschans lock at Antwerp, the great new sea-lock at Ymuiden (the largest in the world), and elsewhere? It seemed to him that the method of lowering the water-table outside a sheet-piled enclosure, thereby reducing the hydrostatic pressure on the underside of an impervious foundation stratum, might have been of valuable assistance at Calcutta, if the conditions did not render its application impracticable. The interior chambers of those important lock entrances, including Bremerhaven, had no pressure-resisting floors. At Ymuiden the natural floor was simply levelled with a coating of small rubble to guard against the effects of wash and propeller action; while at Antwerp the thin concrete covering was perforated with vent-holes.

Mr. Du-Plat-Taylor.

Mr. F. M. G. Du-Plat-Taylor thought that some of the alterations might have been decided upon at an earlier stage. For instance, the raising of the level of the floor of the lock from − 30 feet to − 22 feet was stated to have been due to the fact that a level of − 22 feet

[1] Minutes of Proceedings Inst. C.E., vol. 226, p. 178.

Mr. Du-Plat-Taylor.

was all that could be obtained over various shoals in the channel of the river. That could not have been a new condition; but perhaps extensive dredging in the river had at one time been contemplated. The diversion of the quay-wall on account of the presence of the mosque has spoilt the lay-out of the main dock and limited the berth at No. 3 shed to about 550 feet, whereas, if the quay-line at No. 2 shed had been carried straight through, there would have been a berth of more than 700 feet. About 400 feet of unproductive quay-wall had been constructed on a curve, and no one would say that native religious susceptibilities had not been regarded to the full. The berth at Export Shed A was very close to the dry-dock entrance, and it appeared likely that ships lying at that berth would have to be unmoored and moved temporarily when vessels were being docked or undocked in the dry dock. It would have been better to recess the quay line at " A " berth by about 50 feet, so as to avoid such interference. The final arrangement of the caissons in the lock and dry dock seemed to be excellent, but he had grave doubts whether the latter could ever be used as a lock in case of emergency. The same arrangement had been made in the design of the Tilbury Docks, built in 1886, with the same object; but in the course of 45 years the use of the dry docks as a lock has been tried only once, and the experiment had never been repeated, being regarded as dangerous, owing to the time taken to remove and replace the ship caissons. In Fig. 8, Plate 8, small monoliths were shown below the front stanchions of the sheds. No piles were shown, but it was assumed that all the other column bases were so supported. That seemed to be a bad plan, for the resistance of those two forms of foundation must be very different, and any slight subsidence of the pile-supported stanchions would result in deformation of the shed structure between the monolith and the pile foundations, with resulting roof leakages and cracks in floors. The costs given were interesting. The excavation inside monoliths worked out at about 4*s*. 3*d*. per cubic yard; brickwork in cement in monolith walls 30*s*. per cubic yard, and brickwork in superstructure walls 24*s*. 9*d*. per cubic yard.

Turning to Mr. Pearson's Paper, he agreed that the rendering of the faces of monolith walls sunk through clay was useless. The action, in his experience, was that the monoliths in sinking sheared off a layer of clay and carried it down, the friction being to a large extent that of clay on clay. The skin friction—which averaged 448 lbs. per square foot, but in one case was as high as 1,052 lbs.—agreed fairly well with the 855 lbs. recorded at Tilbury[1] in 1916.

[1] F. M. G. Du-Plat-Taylor, "Extensions at Tilbury Docks, 1912–1917." Minutes of Proceedings Inst. C.E., vol. ccxv (1923), p. 165.

Mr. Du-Plat-Taylor.

No average rate of sinking was stated, but no doubt that would be governed by the rate of building up the monoliths, which was always slower than precast blockwork. Much lighter plant had been used than would have been necessary with blockwork, the capacity of the cranes being limited to 3 tons. That had involved providing the kentledge in the form of small blocks of 2·36 tons, which must have entailed much extra handling to obtain the required total weight; and a great deal of time would no doubt have been saved by employing 5-ton kentledge blocks and 5- or 6-ton cranes.

Mr. Milne.

Mr. A. G. Milne observed, with reference to Mr. Ash's remarks on the altered design of the quay-walls of King George's Dock, that it would be interesting to know what angle of repose had been taken for the earth, the weight per cubic foot of the material, the surcharge allowed, and the assumptions made as to water-levels in the basin during construction. The use of the factor $(1 + \sin\phi)/(1 - \sin\phi)$ in calculating the earth resistance in front of the wall would of course indicate a more stable wall; but was it safe to assume that the passive earth-pressure in front of the wall would always be acting with its maximum intensity? Had the decision to increase the width of the wall from 36 feet to 44 feet been made because the calculations indicated excessive toe pressure on the foundations, as would appear likely? In what way exactly had the 36 feet wide wall been judged to be unstable? What had been considered to be the maximum permissible pressure on the foundations? With regard to the tendency for the wharf structure at Garden Reach to move riverwards, had this movement occurred after completion of the structure or during construction? Why had the form of flooring adopted in the Garden Reach sheds been abandoned in favour of a reinforced-concrete slab floor on steel joists in the case of the King George's Dock sheds? Steel troughing filled with cement concrete and with a good concrete top formed a floor of great durability, which in his opinion it was difficult to improve upon.

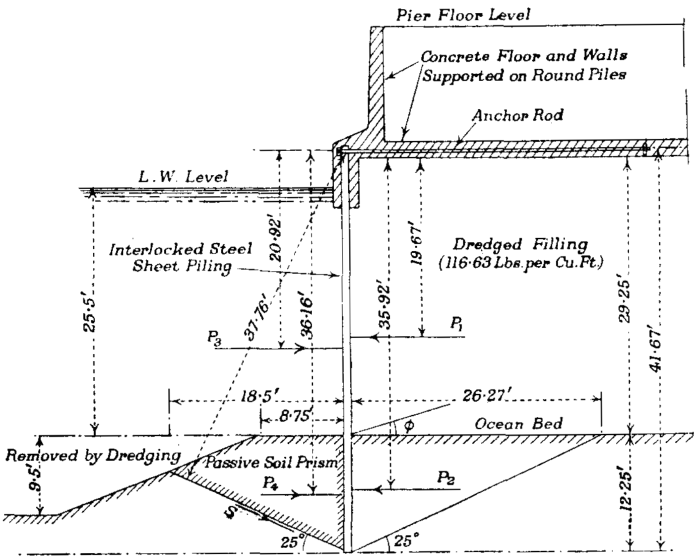
Mr. Rothery.

Mr. S. L. Rothery remarked that Mr. Ash preferred to use the active factor $\frac{1 - \sin\phi}{1 + \sin\phi}$, rather than the passive factor $\frac{1 + \sin\phi}{1 - \sin\phi}$, for the 18 feet of earth resistance below dock-bed level, surcharged with the weight of 36 feet of water above. In a previous discussion[1] Mr. Ash had raised the question whether it would be justifiable to assume that the inverse ratio might be adopted for computing passive pressures. A recent failure at Long Beach, California, of the "flexible" type of quay-wall described by Mr. R. N. Stroyer,

[1] Minutes of Proceedings Inst. C.E., vol. 226 (1929), p. 175.

Mr. Rothery. had led Mr. Rothery to consider the apparent justification for inverting the factor, which gave a passive resultant of two to ten times the value ascertained with the active factor for values of ϕ ranging between 10° and 30°; such angles of repose were permissible for sand or clay soils consolidated with a surcharged weight of 30 to 40 feet of water. Inverting the factor when the same ocean bed conditions existed on inner and outer faces of the wall (as when steel sheet piling was driven into the bed) apparently introduced a risk into the design, because the magnified passive

Fig. 1.



resultant force produced a factor of safety of stability which did not exist for moments calculated about the anchorage at low-water level. Any soil wedge against either face of the sheet-pile wall could have but one resultant force value, and it certainly seemed an erroneous deduction to change a sign in the expression for that force before arriving at its value, merely because there were forces acting on the opposite side of the wall. Text-books treated of the deduction of the active factor, leaving the student to figure the passive resistance from a complexity of unit and conjugate stresses, or ellipses of stress. He recommended the same formula, using the value $(1 - \sin \phi)/(1 + \sin \phi)$ for the passive force as well as for the active force, thus arriving at all horizontal soil forces by the same formula—which would readily be acknowledged as logical—then solving separately the resistance force along the angle of repose, which force Mr. Rothery. was simply the product of the combined consolidated weight of the passive soil unit, due to its own weight and the surcharged weight of water, and the sine of the angle of repose. That force was the shearing resistance along the angle of repose (strictly speaking, the complement of the angle of rupture should be used, which was somewhat larger than the angle of repose). The moment of the resisting force about the same anchorage acted in conjunction with the passive forces, determining whether the structure had a factor of safety or was unstable. The soil along the shearing plane had a definite cohesive or shearing resistance per cubic foot, which must be known and not exceeded when the resisting force was divided by the length of the plane. With regard to the Long Beach failure to which he had referred, *Fig. 1* showed the dimensions and positions of the forces acting on a sheet-piling wall 41·67 feet deep; at one of the critical points, P_1, P_2, P_3, and P_4 had been computed by the Rankine formula, and S had been computed as already stated. The weight of the interlocked sheet piling was 31·75 lbs. per square foot. The filling-material was sandy mud dredged from the ocean floor, its mechanical analysis being—

Passing 200-mesh sieve	57·8	per cent.
,, 100- ,,	36·0	,,
,, 80- ,,	3·2	,,
,, 50- ,,	1·5	,,
,, 40- ,,	0·5	,,
,, 30- ,,	1·0	,,
	100·0	,,

Its weight dry was 85·4 lbs. per cubic foot, with 48·8 per cent. of voids, which, when replaced by sea-water, added 31·23 lbs., making the effective weight 116·63 lbs. per cubic foot if the buoyancy of the water in the filling had no effect. The material showed a cohesive property, and when containing only 20 per cent. of water would stand with vertical sides to a height of 2 feet or more. With 36·4 per cent. of water it was in a semi-fluid condition, so that with a saturated condition of 48·8 per cent. its angle of repose was undoubtedly zero when the factor $(1 - \sin\phi)/(1 + \sin\phi)$ equalled unity. When ϕ was zero the "liquidity factor" [1] was taken into account. Because of the tidal range and the interlocked piling, the filling above low water would retain the moisture in the voids, and with this new type of construction there was no surcharge on the filling. By applying the usual calculations for the horizontal forces

[1] Minutes of Proceedings Inst. C.E., vol. 226, p. 123.

Mr. Rothery. when P_4 was the passive resistance found by the inverted factor $(1 + \sin \phi)/(1 - \sin \phi)$, the following values were obtained:— $P_1 = 49{,}892$ lbs. when $\phi = 0°$ and 25,919 lbs. when $\phi = 18° 26'$; $P_2 = 20{,}521$ lbs., $P_3 = 20{,}808$ lbs., $P_4 = 64{,}451$ lbs. Their moments gave an exaggerated factor of safety of 1·6 to 2·2 for values of ϕ from 0° to 18° 26′ of the filling, and the shearing force against the toe prism was unknown. Applying the similarly-calculated horizontal forces against the piling, P_4 appearing as a similar soil force to P_1, P_2, P_3, and calculating the resistance force, S, acting along the angle of repose, $P_1 = 49{,}892$ lbs. and 25,919 lbs., $P_2 = 20{,}521$ lbs., $P_3 = 20{,}808$ lbs., $P_4 = 10{,}615$ lbs., $S = 19{,}154$ lbs. The moments of those forces showed failure when $\phi = 0°$, giving a factor of safety of only 1·2 when $\phi = 18° 26'$, and as the length of the plane of rupture was 19·5 feet, the soil had to resist a shearing stress of 19,154/19·5 or 980 lbs. per square foot, which was excessive for soil and might have accounted for the failure. Sudden outbursts occurred in several places, bending the steel sheet piling wall outward and releasing the soil behind the wall. The distortion of the piling was evidence of excessive active moments, or lack of soil cohesion, for the length of piling used.

Recent research [1] showed that consolidation and time had material effect in increasing the shear resistance of soils. The weight of the triangle of active soil in situ, due to the surcharged filling, was 674 lbs. per cubic foot on 161 square feet, or 108,031 lbs. The weight of the prism of passive soil *in situ*, due to the surcharged water, was 349 lbs. per cubic foot on 130 square feet, or 45,370 lbs. Even if an internal coefficient of friction as high as 0·4, because of consolidation, were adopted, the resistance to shear along the passive plane of rupture would be $45{,}370 \times 0{\cdot}4 = 18{,}148$ lbs., which, being less than the force S of 19,154 lbs., showed that the soil had insufficient cohesive resistance, should such a high coefficient be proved acceptable. He believed that when P_4 was calculated with the pressure factor $(1 - \sin \phi)/(1 + \sin \phi)$, and the passive resistance, S, applied to the toe of rigid piling was separately calculated from the consolidated and surcharged weight of the passive prism, the solution was logical and more readily understood. The toe of the piling should be at such depth as to ensure a factor of safety of 2 to 3 by calculation of moments of all five forces about the anchorage; and the cohesive resistance of the soil (usually 300 to 700 lbs. per square foot for clays) should not be exceeded by the value of S divided by the length of the plane of rupture.

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[1] Proc. Am. Soc. C.E., vol. 58 (1932), pp. 639 and 640.

Mr. F. W. H. STILEMAN observed that Mr. Ash gave no information as to the cargo tonnage handled at the riverside berths since completion, or statistics to show what the growth in the trade of the port had been; nor did he indicate the period for which the wharfage now provided in King George's dock was estimated to be sufficient. Whereas the five riverside berths had cost £1,983,575, the five berths within the dock had involved a total outlay of £7,115,850. A very considerable portion of the capital expended on the entrances must therefore be unproductive until the trade of the port justified further development of the dock system. From the figures given on p. 327 it was probable that fully 90 per cent. of the vessels trading to the port could be moored at the riverside berths, while the depth of water provided was more than adequate for any such vessel. While recognizing the disabilities of a riverside berth, subject to a substantial rise and fall of tide, it would, on the information given, seem reasonable to think that extended facilities of that nature would have been a more economical course than that actually adopted. On p. 324 it was stated that various reasons existed against the utilization of the foreshore between the Calcutta jetties and Kidderpore docks for jetty accommodation. It would be of interest to know whether they were engineering reasons or reasons put forward by navigation authorities. Experience tended to show that navigation objections were not infrequently pressed without full knowledge of the largely increased capital expenditure which acceptance would involve. It was stated that, owing to the type of construction adopted for the riverside berths, it was intended to provide moorings independent of the jetty structure. Could Mr. Ash give any details of the moorings so provided? The growth in lorry traffic was in accordance with experience in Australia and, no doubt, elsewhere. At Fremantle nearly 30 per cent. of the total cargo tonnage handled reached or left the wharves by motor-lorry, for which reason road access to the sheds had, in recent years, been much improved, and additional space between sheds, for sorting purposes, had had to be provided. It would be of interest to know what regulations had been made and what, if any, other steps were taken to ensure that the low permissible shed-floor loading of 2 cwt. per square foot was not exceeded. In his experience much heavier loading per square foot was usual at general-cargo sheds; while at wharves where bagged wheat was shipped bags were usually stacked to a considerable height. The Australian standard 3-bushel bag gave a load of approximately 40 lbs. per square foot for each bag high. Mr. Stileman.

Mr. ASH, in reply, stated that the suggestion put forward by Dr. Mr. Ash.

Mr. Ash. Cunningham, regarding the alignment of the northern wall of the basin, had actually been considered, but it had been found to affect the spacing of berths along the eastern quay. The Circular Garden Reach Road had, in any event, to cross the dock system, and it had to pass on some convenient alignment between two berths. Dr. Cunningham's estimate of £55,000, as representing the loss sustained by the Port Commissioners on account of the mosque, was very much nearer the mark than the figure of £500,000 put forward by Sir Clement Hindley in the discussion. As to the possibility of lowering the water-table by means of filtration wells, that system had come to Mr. Ash's notice, but at too late a stage for it to be given serious consideration. In any event, its application to the running sand stratum in Calcutta, for the purpose of forming the floors of the entrances, would have been a serious step to take. It would be recalled that the only protection against the water of the river on the one side and of the basin on the other had consisted of earth "bunds," and any disturbance at low levels beneath these might have had very serious consequences. Mr. Ash considered that the best course to take for the formation of the floor was that actually followed, though it was admitted that it had not at one time seemed a practicable one. Advantage had been taken of circumstances which, for once, were favourable to the engineers. He was much gratified by the interest which had been displayed in a question to which he had drawn attention, namely that of the design of quay-walls. He felt that that matter was one of the highest importance to port authorities. Port charges, and consequently distribution costs generally, must under existing conditions be sensibly higher than they need be, and development might be seriously retarded in future, unless engineers could contrive cheaper methods of construction. He had been much struck, during a tour of inspection of certain African ports, by an adherence to methods which seemed to him needlessly extravagant, and he could not see how such berths would pay their way. The same remark might apply to the quays at King George's dock. At the same time everything depended, as had been realized by members whose views had been received, on a sure knowledge of the behaviour and characteristics of the soils in which such works had to be constructed, and it did not appear that any very great advance was being made at the moment. Given some assurance on those points the preparation of an economical design was not a matter of great difficulty. In his reply to the discussion (p. 388) he had recorded his belief that, in the present state of knowledge, the conduct of a large-scale experiment, somewhat on the lines of those carried out by him (p. 351), would be justified, whenever costly works were in contemplation.

It would, he considered, narrow the field of conjecture, although results would always need careful interpretation and the need for judgment and discretion would not be eliminated. At the same time, he would, if ever again engaged on such a test, endeavour to improve on the methods employed. An experiment such as was then conducted, could give information only regarding active forces, and a knowledge of the passive forces was also desirable in order to grasp the real position. He would throw out the suggestion that any one undertaking such an experiment in future might go one step farther and endeavour, by means of hydraulic pressure, to ascertain the forces necessary to reverse the deflection of steel beams distorted by active earth pressures. His answer to Mr. Milne's question whether it was safe to assume a passive force as acting against the toe of a wall was, firstly, that the case for the simple assumption of an active force was untenable, since it led to the conclusion that a shallow wall must of necessity and without regard to changes of strata be more stable than a deep one. Next, the fact had to be recognized that those forces could not act without there being motion, whether perceptible or not; and as soon as that happened circumstances must alter. Whilst the stress against the toe might, up to a certain stage of construction, only be that of the active conjugate force set up by the weight of the earth above, this must inevitably be increased as soon as motion, though ever so slight, occurred. He thought the conditions could be compared to those of an ordinary foundation bearing a vertical load. An "active" upward force of nil tons per square foot would be transformed, during loading, to a "passive" force of whatever intensity the surface might be able to bear. That analogy could not be pressed further, but it might explain Mr. Ash's conception of what would occur at the toe of a quay-wall. Mr. Rothery had, in his opening quotation, inverted the view expressed by Mr. Ash, who would also like to make clear that he regarded the factors $(1 - \sin \phi)/(1 + \sin \phi)$ and $(1 + \sin \phi)/(1 - \sin \phi)$, in themselves, as devoid of significance when used in relation to clayey soils; but if a soil were of such a nature that the pressures exerted by it were judged to be capable of being arrived at by Rankine's theory, then he saw no alternative to the use of the factor $(1 + \sin \phi)/(1 - \sin \phi)$ for deducing the pressure in front of a wall. It might be simpler to use for the purposes of this discussion a factor k, less than unity, as applied to the vertical stress behind a wall to denote the active conjugate stress; thus $P = kw$. Presuming that a value for k had been ascertained by deductions from the results of an experiment on a scale altogether beyond that of the laboratory, a case might be argued for adopting Rankine's general theory of the relation Mr. Ash.

Mr. Ash. between active and passive stresses, whilst disregarding his methods of computing those stresses. Then, if the vertical stresses in front of the wall were represented by W_1, the passive conjugate stresses in front could be visualized as rising, during loading, from just over kw to W_1/k. He did not wish to suggest that a pressure uniformly increasing from dock-bed level to bottom of wall and calculated on the latter formula would be a safe assumption to make: there were many more factors to be considered. He had no general proposition to propound, but quay-walls were a special case. There was about quay-walls, built for ocean-going ships, a degree of uniformity which made it possible to consider such walls as in a class by themselves. The calculations made for the King George's Dock quay-walls contained the inherent assumption, not perhaps realized at the time, that the earth-pressure on the face of the wall just below dock-bed level was only 37 per cent. of the water-pressure just above, and further, that the increase of intensity per foot of depth below that was less than that which would be due to water. Had the dock been dredged out to the level of the toe of the wall, the wall would have appeared, so far as the figures went, to be very much more stable. He had analysed the conditions applying to certain existing walls and had found that they could not possibly be stable if such were the nature of the stresses. He had concluded that the earth-pressure just below bed-level must be at least equal to that of the water just above, though it need not necessarily exceed it. The distribution of stresses below that level was admittedly indeterminate at present, but the method of treatment for "flexible" walls which he had worked out for himself, though too lengthy to describe in this connection, was based on the belief that the passive conjugate earth-pressure

(*a*) might at bed-level be taken as equal to that of the water above;

(*b*) would increase to a maximum not exceeding W_1/k at a point between bed-level and the bottom of the piles; and

(*c*) would diminish below that point, though not necessarily to zero, at the bottom of the piles.

As an instance, this would indicate, in a case where $k = 0{\cdot}5$, a pressure equal to twice the vertical pressure, acting on the face of the piles, but purely locally. The question of the *vraisemblance* of such a method could be decided by working out various cases; it was not susceptible of direct proof. He would not venture to apply such reasoning to the Long Beach wall, of which Mr. Rothery gave particulars, on account of the deepening in front of the berth, which would seem to render the distribution of stresses more problematical than ever. The section seemed to be a very hazardous one

Mr. Ash.

to adopt, though it was noted that Mr. Rothery attributed failure to under-estimation of active, not passive, forces. It would have been interesting to know whether there had been movement at bed-level. No account had apparently been taken, in the computations, of the support afforded by the anchor-rod. It would be regrettable if failures such as that set back the clock. Mr. Rothery's method of computing toe resistances seemed preferable to that of assuming a mere active pressure, but Mr. Ash was at a loss to see its justification. It appeared to take into account the same force twice over. The value $(1 - \sin \phi)/(1 + \sin \phi)$ was of itself nothing but an expression for the horizontal reaction due to a prism of earth; but under this method of computation there was added to it the reaction from another and a bigger prism, comprised in the same mass. Mr. Ash found difficulty in accepting that. In reply to Mr. Milne's further queries, the calculations of the King George's Dock quay-walls had been based on an assumption of an angle of repose of 80° 36′ and the 36-foot wall had been found to be liable to overturn. Mr. Ash believed that a specific angle of repose, when applied to such soil, was almost meaningless. It would stand at all sorts of angles, according to the conditions, and when there was a slip it was generally along a surface which was approximately parabolic. The toe pressure of the 44-foot wall had under the method adopted been calculated at 5·3 tons per square foot.

The disadvantage which Mr. Du-Plat-Taylor apprehended in connection with a vessel lying at berth "A," when another ship had to enter the dry dock, had not been experienced in practice, and room enough had in point of fact been left. The question raised by him of the use of the tandem graving-docks as an emergency lock had been touched on by Mr. Reed in the discussion. Mr. Ash agreed with Mr. Du-Plat-Taylor that the need for this was very unlikely to arise. But so much capital had been made of the idea as a justification for the construction of the two graving-docks parallel to the entrance that it had been thought advisable, when it was desired to remodel the caisson system, to make a point of demonstrating its feasibility under the new arrangement. The methods adopted for carrying the loads of the dock transit shed foundations had been described on p. 357. The disadvantage of having different types of foundation had been realized, but whilst monoliths over the whole area were out of the question on the score of expense, concrete piles just behind the wall, in earth recently subjected to disturbance from monolith-sinking and liable to settlement along inclined planes, would have been too risky altogether. Mr. Milne's query about the outward tendency of the riverside jetties had been answered in the reply to Mr. Reed's observations in the discussion. Mr. Ash was indebted to Mr. Milne for calling attention to the fact that the dock

Mr. Ash. transit shed floors had been shown on Fig. 8, Plate 8, as of reinforced concrete. This draughtsman's error had been overlooked; the floors were of steel troughs, filled with cement concrete.

The cargo tonnages handled at the riverside berths, asked for by Mr. F. W. H. Stileman, were :—

Year.	Imports 1,000 tons.	Exports 1,000 tons.	Total 1,000 tons.
1924–25	123	287	410
1925–26	143	303	446
1926–27	120	316	436
1927–28	154	359	513
1928–29	250	362	612
1929–30	195	316	511
1930–31	148	254	402
1931–32	118	192	310
Total	1,251	2,389	3,640

These figures dated from the year in which the last of the four produce berths came into use. They did not include coal shipped at the upstream jetty. He begged to be excused, in the present state of trade, from making any prediction regarding the period over which the present wharfage at King George's dock would be likely to meet the needs of shipping. The number of riverside berths which could be provided on the Calcutta side of the river was limited by the length of foreshore fringing the "cutting" edge. There was shoal water to the downstream side of the entrance to King George's dock. Between the riverside berths and Kidderpore docks lay property which it would have been very costly to acquire, whilst the greater part of the foreshore between Kidderpore docks and the Calcutta jetties fronted the "Maidan," a large open space and recreation ground, open to the public except for the area occupied by Fort William. Jetties along that foreshore were out of the question, for military and public-health reasons.

Riverside jetty moorings were carried at the face of the jetty by three steel screw-piles, the centre one of which was vertical and the upstream and downstream ones raking. These piles were united, just below deck-level, by a strong cast-steel cap, bearing with a flat surface against the horizontal timber fender. The cap carried two swivelling shackles in front to take ships' cables, and it also had two lugs at the rear, to which were attached the upstream and downstream ties, each aligned at about 45 degrees to the jetty face. The effect of a pull from a ship's cable was to put one tie in tension and to give an inward push to the face of the jetty. Each tie consisted of two 12-inch by $3\frac{1}{2}$-inch channels, back to back, and they extended to not less than 80 feet behind the jetty proper. At the shoreward end of each tie there was a grillage of short 10-inch by 5-inch rolled steel joist piles, and the pull from the tie was distributed on to these piles by a cross beam. The grillage was bolstered up, on its river side, by a mass of concrete, almost coffin-shaped on plan, 10 feet thick and 26 feet in length. The whole arrangement, if somewhat elaborate, was sound in principle. The question of the superficial load on the riverside berth shed-floors had been dealt with in his reply to the discussion. Mr. Ash.

Mr. PEARSON, in reply, remarked that he was glad to see that Mr. Du-Plat-Taylor agreed with him that rendering the faces of monoliths sunk through clay was useless, as that was not the universal opinion. The 3-ton cranes were quite as heavy as Mr. Pearson would have cared to use during the monsoon in the treacherous soil at King George's dock. Mr. Pearson.