every foot in order that the pipe could be raised or lowered to suit the water-level.

ACKNOWLEDGEMENTS.

The Author wishes to express his thanks to Messrs. J. Lomax Simpson, S. Woolley, and G. B. Lissenden, and to Messrs. Holland & Hannen and Cubitts, Ltd., for permission to reproduce illustrations.

The Paper is accompanied by fourteen sheets of drawings and twenty-four photographs, from which the folding Plates, the half-tone page Plates, and the Figures in the text have been prepared.

Discussion.

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

The Chairman said that, during the comparatively short time that the Structural and Building Engineering Division had been in existence, the Papers presented had ranged from the purely theoretical to the essentially practical. The Author's Paper was certainly in the latter category. The works described were not on the large scale ; in fact, it had been emphasized that they were on the small side. The value of the Paper, however, did not lie in the magnitude of the works described therein, but rather in the account given of the special difficulties encountered, and the Author was to be congratulated on giving such a frank account of the setbacks and misadventures that had occurred in the course of some of the works. He particularly liked the human touch in the passage where the Author explained how he had been discomfited by the inadequacy of a 3-inch pump.

Mr. W. C. Andrews observed that he was sure that the information given by the Author would be useful to many members, and he hoped that many more Papers of a similar character would be presented to the Division.

With regard to the first two projects described, the Author had stated that provision was made not only for letting the effluent out under gravity but also for pumping it. Mr. Andrews had noticed that there was an open penstock chamber, and he was at a loss to understand how the Author was able to pump the sewage through a pipe with what appeared to be a break in it.

In dealing with the question of the load on piles, the Author had stated, in regard to the necessity of making actual tests rather than using design formulas, that it was his practice to design in accordance with the known conditions. That again was not quite clear to Mr. Andrews, because he believed that in the various more recent formulas that were used an endeavour was made to take known conditions into account.

The Author had also stated that to stiffen the jetty the piles, particularly under the cranes, were hacked and encased in cement. That work was very close to the sewage outfall. What kind of cement was used—aluminous or ordinary Portland cement? There would appear to be considerable danger that the effluent—especially the sewage—would have a harmful effect upon the concrete.

The buffers or fenders that were provided appeared to be just strapped on, with no provision for springing in the event of barges or ships tying up; and they seemed to him to be far too rigid.

Mr. S. Packshaw observed that he would confine his remarks to the canal and river walls described in the Paper. He wished that more information could have been given about the nature of the subsoil. The Author had expressed a dislike for "dead men," by which, presumably, he meant all kinds of anchorages—not merely mass concrete blocks, but also piles, and so on. Doubtless he accepted anchorages as unpleasant necessities, because in most cases it was impossible to make a retaining wall of sheet-piling stable without the use of anchorages of some kind. An unanchored wall was relatively heavier and more expensive than one which had been tied back, for the same reasons that a cantilever beam was heavier and more expensive than a supported beam. It was difficult to combine relative economy in even a small unanchored wall with the same factor of safety as could be provided in a tied wall, especially when account had to be taken of unexpected superloads, such as stacks of coal or timber. Doubtless the Author, in his very wide experience of riverside structures of all kinds, had seen many failures which could be ascribed to anchorages in some form or other; but Mr. Packshaw submitted that it was not the principle which was at fault in such cases, but the designers. The most frequent cause of failure was placing the anchorage too near the wall. As soon as the wall yielded a little under the influence of the earth-pressure behind it, an unstable wedge tended to form behind the wall, and as that slipped forward it took the anchorage with it. Slightly longer ties, adding very little to the cost of material or excavation, would enable the anchorage to be placed in stable ground and would overcome the difficulty. Sometimes lack of space did not permit ties of sufficient length to be installed; in that case a very substantial anchored wall was necessary, driven well down into the stable ground below the wedge. In other cases anchorages consisting of single timber piles or sheet-piles were employed, but, as a rule, they were too narrow to develop sufficient resistance from the ground in front of them and were therefore liable to fail by shearing through the ground. Another procedure was to form an A-frame consisting of front and back raking piles and to ensure that there was a substantial vertical load on top of it, so that the pull in the tie-rods was taken up by compression in the front rakers and tension in the back ones. That method, although very satisfactory, was rather expensive, and it was not generally used except with very high walls or if the ground was very poor. If the subsoil was reasonably good the most effective type of anchorage was a simple concrete block or an anchored wall of some kind, located in the stable ground well away from the unstable wedge. That was the usual practice, and a typical example was illustrated by Fig. 7, Plate 2. It would have been even better if the concrete block in front of the steel anchor piles had been omitted, because that would have enabled an even greater distance to be obtained between the face of the anchorage and the wharf itself.

Another cause of anchorage failure was the use of tie-rods which were too small, especially if corrosion was taken into account. Many engineers were of opinion that steel which had been well coated with tar and wrapped with tarred hessian did not corrode a great deal if it was permanently embedded in the ground, and Mr. Packshaw himself had seen tie-rods in very good condition after having been in the soil for 70 years or more; but he considered that some initial corrosion must take place, and obviously a reduction of, say, $\frac{1}{16}$ inch in the diameter of a small tie-rod was much more serious than an equal reduction in the diameter of a larger rod.

With regard to the ingenious design for reconstructing the timber wall illustrated in Fig. 10, Plate 2, he had tried to make a rough guess at the quantity of steel involved in the joists and the reinforcement of the riverside piles, and he had estimated that, without the reinforcement in the slab, it amounted to about 600 lb. of steel per foot of wall. If the Author could say what height the wall was supposed to retain, that would enable some comparison to be made between the time and cost of the method adopted and the time and cost of re-facing the whole wall in either steel or reinforced-concrete sheeting.

Mr. F. M. G. Du-Plat-Taylor said he was very glad that the Author had described a number of medium and small works, because they belonged to a class which had been largely neglected in the past. At one time it had not been considered proper to present to The Institution any Paper describing works costing less than £500,000. Consequently many descriptions of smaller works had been submitted to other Institutions and Societies, and he thought The Institution had lost thereby.

He agreed with the Author as to the necessity of carrying out loading tests on piles instead of relying on the various formulas; that view had also been emphasized in the Paper presented recently by Mr. H. D. Morgan, M. Inst. C.E.[1]

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[1] "The Design of Wharves on Soft Ground." Journal Inst. C.E., vol. 22 (1943–44), p. 5 (Mar. 1944).

The canal walling or camp-sheeting shown in Fig. 5, Plate 1, and *Fig. 6*, which was made out of the cut-off from the main works, did not seem adequate to carry a heavy surcharge, such as the large stacks of timber shown in *Fig. 6*. No information was given about the subsoil. If the piles were driven into the ballast, the wall would appear to be adequate; but if they were merely driven into clay he thought that a surcharge as heavy as the stacks of timber shown would be too great.

The statement, on page 6, that the Larssen piling in the river wall was anchored back to "dead men," which in turn were securely anchored to the reinforced-concrete road slab, was not clear, because the tie-bars were sloping downwards and therefore when they reached the anchor blocks they would be 10–12 feet below the road slab. He did not see how the connexion was made between the two.

The design of a river wall, utilizing the sound parts of the old camp-sheeting and suspending the lot from the deck-work, seemed very ingenious. Again the nature of the ground was not mentioned, nor was the depth of the water alongside given.

The design of the pre-cast decking for the pier shown in *Fig. 11*, Fig. 12, Plate 3, and *Figs. 13* and *14*, seemed admirable. Renewal of decking was always expensive, particularly on pierheads outside dock entrances, which were usually of timber. When the deck-bearers were of wood, it was important that the decking should be continuous where it crossed the bearers, as otherwise water lodged and decay set in in the bearers. Wooden decking was generally spaced $\frac{1}{4}$ inch apart, and packing-pieces had to be inserted where it crossed deck beams; but, with the design of pre-cast decking illustrated, the decking was wider where it rested on the deck beams and therefore the deck beams were protected against moisture.

The use of old ships' funnels for carrying temporary staging was a bright idea, and would qualify for the "Ingenuity Competition."

Was any special treatment applied to the stairways to prevent slipping? Stairways to half-tide or low-water landings were always covered with slime. He remembered a case in which nails, of a type similar to those sometimes used on the soles of boots, were driven into timber steps, and proved very effective.

On the largest work described in the Paper—the extension to an existing dock—the scheme for a dam consisting of the rock itself plus a line of steel sheeting was very ingenious. What was the fragmentation of the sandstone rock? He had had a good deal of experience in Liverpool with sandstone rock and he knew that it tended to blow into very large pieces, which had to be broken up with rock-breakers before they could be removed, whereas harder rocks blew into much smaller pieces.

Mr. F. W. Davenport expressed his thanks for the invitation to participate in the discussion. He felt that it was a gesture which symbolized the desire of the profession to co-operate with the manufacturers in the gigantic task of reconstruction which faced everyone in Great Britain.

He had noticed that for the pump-house described on p. 4, *ante*, 15-inch by 5-inch universal section steel piles were used. A tendency existed to regard universal piling as a rather uneconomic section, and he would be glad if the Author would explain why it had been adopted.

Rapid strides had been made during the past 10 years in the use of steel sheet-piling—chiefly the one-piece Z-shaped sections, and it had largely taken the place of mass concrete or masonry retaining walls in dock and harbour construction. In deep excavations for heavy foundation work it offered a reliable form of shuttering, answering the dual purpose of retaining the earth and keeping back subsoil water. The economy obtained from the use of that particular type of section, in comparison with the older type, was due to the greater modulus which was given for a considerably less weight of steel, together with a consequent considerable saving in timber walings and struts.

One of the chief advantages of steel sheet-piling was the rapidity with which work could be carried out, in comparison with older methods, and he thought it would undoubtedly play an important part in the work of reconstruction. He had chiefly in mind the rebuilding of docks and harbours on modern lines and the cutting of new waterways and improvements to rivers.

In many instances the manufacturers found it extremely difficult to obtain adequate information in regard to the strata, such as the depth of the various formations and their properties, which was vital to the designer. Without adequate information a job could not be economically designed, and in his opinion it was well worth while to spend a few hundred pounds on site-testing in the initial stages.

He could give an assurance that the heavy steel makers were prepared to offer wholehearted support and co-operation in the formidable tasks facing the civil engineering and building industries.

Mr. J. G. Arthur observed that the works described in the Paper were mainly of an industrial character, and the problems were of the kind which many—perhaps most—engineers had to face at present. They involved one vital necessity, namely, the making of snap decisions when new circumstances arose. He imagined that in larger works there was less need for very rapid decisions, as such jobs were carefully considered and accurate forecasts were made of what was likely to happen.

In the first work described the Author had used pre-cast reinforced-concrete piles, which were driven to normal sets and carried loads of the usual values, and he had expressed the view that actual loading tests should always be made. About one hundred formulas were available for calculating the supporting power of piles, which yielded results ranging from very low to very high values; and, whilst a few of those formulas gave useful guidance, they should be regarded only as guides and not as proof of the bearing power. Many cases had occurred in which piles apparently driven to a good set had nevertheless failed under a constant load.

Practical experience in driving, coupled with some guidance from a reasonable formula, followed by a loading test on an actual pile, was necessary to ensure that the piles would perform their duty, and the loading test was of primary importance. Any such loading test should be in the form of a dead load test if possible, and care should be taken to ensure that there was no possibility of the test-load being applied eccentrically or tilting during the course of loading. If it was impracticable, owing to site congestion or other reasons, to apply a dead load test, a jack test might be used, possibly employing adjacent piles to obtain the necessary anchorage; but very careful supervision was necessary to ensure maintenance of the full jack load without variation for a considerable period. Jacks were rather wayward instruments at times. After the full test-load had been placed on the pile it should remain for at least 48 hours after any settlement of the pile had ceased.

In the second work described by the Author, the loads were carried partly on Franki piles and partly on pre-cast piles, and the reason for that should be appreciated. The work described formed only a small part of a large contract carried out nearly 10 years ago, in which a contract was made by which 3,200 piles were to be driven to carry 50 tons each, in a period of 14 weeks. The work was carried out in 12 weeks, and a further 1,000 piles were driven for an immediate extension. The piles, which averaged 25 feet in length, were driven at the rate of ten to twelve per day with each machine. In consequence of the general use of Franki piles on the site, their use was continued for the works described in the Paper as near to the river as the Author thought desirable, and pre-cast piles were substituted for the work in the actual bank of the river. That would not now be necessary, as means had been devised and proved of installing Franki piles actually through water; and whilst the process was more expensive foot by foot than the use of pre-cast piles, nevertheless, when only a few river piles were required, as in the case in question, it was cheaper to retain the original type than to change over and send separate plant for driving pre-cast or timber piles.

The piles described by the Author were 18 inches in diameter, reinforced with four $\frac{5}{8}$-inch diameter longitudinal bars and $\frac{1}{4}$-inch diameter helical binding, whilst the enlarged bulbs contained 15 cubic feet of concrete. One pile, which was excavated, showed a bulb of spherical form 3 feet 4 inches in diameter. The driving was comparatively easy through various strata of poor quality to a depth of about 20 feet, when it became harder, the piles finally taking their set in a layer of ballast. They were, in the main, 24–25 feet in length, though occasional areas were discovered where the lengths varied down to 20 feet and up to 28 feet. That, however, did not slow down the work, since no cutting or lengthening was required.

Loading tests were made, but the figures given were the minimum requirements, whilst actually test-loads of 110 tons per pile were applied.

The maximum settlement with the load on amounted to $\frac{3}{16}$ inch, and in two cases the residual settlement was zero and $\frac{1}{16}$ inch respectively.

In three of the works described in the Paper the Author had used "dead men," and was obviously a little worried about anchorage. He wondered whether the Author had considered the use of A-frames, to which Mr. Packshaw had referred, and, if so, whether he had considered the possibility of using piles with enlarged bulbs for those A-frames. In an A-frame one pile was in tension and the other in compression, and the limit to the horizontal pull which such an arrangement could resist was determined by the anchorage value of the tension pile.

Mr. S. Woolley observed, in regard to the extension to an existing dock, that for the final demolition of the existing rock band of sandstone approximately 120 holes, 4 feet deep, were drilled in three series, from the top vertically, from the centre, and from the bottom. The heavy charges were placed in the bottom holes and the lighter charges in the centre holes, whilst a light charge of about 7 lb. was placed in the top holes. All were joined together by "Cordtex," in such a manner that a break in one place, or even in two places, would not affect the final explosion. That being so, it seemed rather surprising that the charge in the bottom holes did not explode, and he would like the Author to explain that.

If the Author had a similar job to do again he might prefer to drill larger holes, say 6 inches diameter—and to drill them 30 feet deep, straight down from the top. The advantages of doing that would be considerable: charging would be easier, only one line of "Cordtex" would be needed, and all the holes could be charged in about half the time required by the method described in the Paper.

Mr. E. N. Horton observed that for periods totalling about 17 years he had been engaged as an architect on the design and construction of industrial premises on three important rivers and one main canal in Great Britain, and he had encountered many small river works of the type described in the Paper.

At the jetty and pump-house, the sheet-piling had its second use for the penstock chamber and outfall. The Author had stated that that sheet-piling had to be cut off at the top to allow the work on the decking to proceed, and that the contractors had credited the scrap value for the piling—which rather implied that at the end of the job it would have to be removed. It was not clear from the Paper how that sheet-piling was removed from under the decking.

On the same work, if he correctly recognized the locality from the section given through the river bank, the low-water level was about 8·75 below Ordnance datum. He had had a very similar case to deal with about 5 years ago, in which similar levels occurred on a mud slope or beach 225 feet long, between low water and the outer edge of the concrete apron at the end of the outfall pipe. That outfall pipe was only 24 inches in diameter, in comparison with the 36-inch pipe mentioned in the Paper, and, apart from general surface water, the pipe also carried circulating water from a factory, the volume coming out of the pipe being fairly heavy. He was not clear whether the pipe in the jetty described in the Paper was used for the same purpose. On the advice of the authority controlling the river bank, the apron of reinforced concrete was constructed in three flat stages, with shallow steps between, and it was also broadened out towards the river, in order to distribute the effluent. Those precautions were not successful, and after a few years a channel about 12 feet in depth and several yards wide, had been carved out in the mud. He would be interested to learn whether the same thing had occurred at the jetty described or whether any successful precautions had been taken to prevent it. The apron was shown only as far as the outer line of piles.

A few more details of the pump-house would be of interest. The floor-level was given as 94·85, whilst the centre-line of the 36-inch pipe was about at datum-level, and that made the pump-house look very tall.

Why, on the jetty referred to on page 4, *ante*, was the gantry made of structural steel rather than of reinforced concrete ? Mr. Horton considered that, particularly in the tidal atmosphere, reinforced concrete would have some advantages from the point of view of maintenance and would possibly be simpler to use.

With regard to the small canal wall (page 5, *ante*), would not the risk of corrosion of the tie-rods have been reduced considerably if they had been encased in concrete ?

*** **Mr. A. S. Grunspan** observed that he would like the Author to clarify the statement made on page 4, *ante*, that if loading tests on piles were not feasible, his practice was to design in accordance with the known conditions. Loading tests formed the surest basis for designing piles ; otherwise an analysis of borings and driving records afforded a good insight, from past experience, into the carrying capacity of the pile, when driven to a given set. Representative borings were often a good guide to determining the required length of a pile.

Some details in regard to pile-casing, forming piers, would be useful ; were pre-cast cylinders sunk, and filled with concrete ? As reinforcement was provided in the casing, and the main object was to take the impact, the load being carried on the pile itself, was it necessary to roughen the pile to form a key for the casing ?

In the canal wall (Fig. 5, Plate 1), the sheet-piling, on which was superimposed a reinforced-concrete wall, was shown driven barely 2 feet below the dredged river-bed. Whilst Mr. Grunspan appreciated that the Author was anxious to make use of sheet-piling recovered from other work, he thought it was important, in that type of construction, to drive the piles to a level at which stability of the ground might be expected. The wharf appeared to be fairly heavily loaded, like most wharves, and although the fender piles,

*** This contribution was submitted in writing.—Sec. Inst. C.E.

driven 4 feet below the sheet-piles, would act to some extent as relieving piles, too much reliance should not be placed upon them, particularly as they were so far apart. Usually, fenders were carried to a level above the feet of sheet-piles. The Author had deprecated the use of "dead men." A number of wharves with timber, steel, and reinforced-concrete sheet piles anchored back to "dead men" had been satisfactorily in use. That system was a cheap way of forming a quay wall, and, provided that the anchor piles or plates were housed in a stable soil, it might be considered a satisfactory form of construction, in certain cases.

With regard to the wall illustrated in Fig. 10, Plate 2, it would be interesting to know why the rolled steel joists were reinforced with bars at top and bottom; were the 24-inch by $7\frac{1}{2}$-inch joists not strong enough without such reinforcement, or were nominal bars introduced for the beam casing? It would be interesting to know why the Author had specifically mentioned that the deck slab was carried between the 24-inch by $7\frac{1}{2}$-inch joists, so as to relieve the horizontal thrust at the top of the wall. The deck must have been so carried, in any case, because the deck span between the 24-inch by $7\frac{1}{2}$-inch joists was 15 feet, whereas it was 33 feet in the other direction.

The thrust at the top of the wall could not be great, and was resisted by the deck, which was of substantial thickness, stiffened by the beams and beam casing.

The Author, in reply, said he would refer Mr. Andrews to Fig. 8, Plate 2, which showed an exactly similar construction to that illustrated in Figs. 1 and 2, Plate 1, in which the tidal flaps and the outfall could be seen. The scheme was that when the effluent ran out on a rising tide, it would be diverted and housed in the tank below the pump-house and pumped out of there either over or through the river bank through the small 18-inch diameter pipe shown on the left.

All the conditions affecting piling could never be known, as in the case of soil mechanics. He did not mind what the formula was; if a pile of the size given in the Paper were driven to the set given it would take that load, in any part of the country; he had proved that by driving thousands of piles. Similarly, if the Franki pile were driven to the set given in any soil it would stand up to the load. That was what he called "known conditions."

The cement used in the case to which Mr. Andrews had referred was quick-setting cement. No special precautions were taken on the jetty. The effluent was not sewage.

With regard to the fenders, the object was to make the construction such that when a boat came alongside it would knock the fender off or alternatively use it as a buffer. The timber resisted the impact on to the actual structure, and if a timber broke it was taken out and easily replaced by a new one, for which provision had been made in the design.

In the Author's experience most failures were due to the fact that the sheet-piles had not been driven deep enough. He agreed that in many cases it was not possible to do without "dead men," but if they could be omitted, by putting in A-frames or anchoring to Franki piles, that was desirable. The conditions varied in every place, and it was necessary to use one's experience and knowledge and adopt what was considered to be the best possible scheme for any particular job.

The height of the wall shown in Fig. 10, Plate 2, was 22 feet from the beach-level to the top of the wall.

The little canal wall to which Mr. Du-Plat-Taylor had referred was what was called an "expedient" job—a cheap job—and much of its strength was due to the timber piling in front. The timber was well tailed back, as shown in *Fig. 6*, and he did not think that much load came on the wall itself. He would not recommend that type of wall for many cases, but it fitted the particular job.

With regard to the anchorage to the road slab (Fig. 7, Plate 2), the part plan showed the No. 2 Larssen steel sheet-piles at the back; those were driven and continued up, so that they in turn were anchored to the road slab, running the full length of the factory opposite the river wall, which was also reinforced, so that there was a really good anchorage, and he did not think it could be called a "dead man."

His own experience with regard to the fragmentation of the rock was that soft rock required more explosive than hard rock. Therefore a larger charge of 1 lb. or 2 lb. of gelignite was put into the soft rock and a smaller charge of $\frac{3}{4}$ lb. or $\frac{1}{2}$ lb. into the hard rock. The fragmentation was small enough for the pieces to be picked up by a No. 27 Ruston-Bucyrus machine; it was a little difficnlt to do that with the No. 14 Ruston-Bucyrus machine, the use of which, as explained, was ultimately dispensed with.

The old British Steel Piling section was excellent for many works. It did not, weight for weight, give the same section modulus as was provided by some others—particularly the Appleby Frodingham pile—but it did afford a solid feeling of security.

Mr. Arthur's firm had offered to drive all the piles for a set figure of cost—a lump sum—and had guaranteed that each pile would take a certain load; if they did not do so, all the plant would be removed and no charge would be made for the work. The firm had carried out the 75-ton tests that were asked for, and then, to please themselves, they had continued to 110 tons and had taken photographs.

The explosive manufacturers had stated that the "Cordtex" would definitely be effective for 12 hours under water; they had known it to be so for 16 hours, but would not guarantee it beyond 12 hours. Therefore, it was necessary to work against time and to try to get the whole of the charge in and blown up before the 12 hours elapsed. The failure of the bottom holes to fire had nothing to do with any breaking of the "Cordtex." The reason was that owing to the first mishap, at 9 p.m., it was not possible to explode until 1 a.m.; thus the bottom holes were flooded much longer than those at the top, and therefore the charges did not explode.

The driving of 6-inch holes vertically downwards and then charging them with explosive from the top in the day was considered at the time, but, owing to the shortage of well-boring and drilling plant of the type required and to the cost and time involved, as well as certain other considerations, that method was not adopted. If he had to do a similar job again with the same conditions, he would drive the holes vertically right the way down, charge them from the top, and then blow under water afterwards.

In reply to Mr. Horton, so far as the apron was concerned he had never heard of any scour causing trouble in the area in question. The sheet-piling underneath the jetty was simply cut off at the deck-level, and afterwards at the river bed-level, and that was why the short lengths were used on the canal wall. He agreed with Mr. Horton that the risk of corrosion to tie-rods would have been reduced if they were cased; in actual fact they were wrapped in hessian soaked previously in liquid tar and again coated.

In reply to Mr. Grunspan, in the Paper the Author was advocating the use of loading tests as against the use of formulas, but if it were not possible to have those tests, then in his experience the information he had given as to the set in driving, coupled with the size and type of pile used, would safely carry the loads indicated, so that, if those conditions were adhered to, it could be said that there were "known conditions" on any site. He did not underestimate the value of trial borings for predetermining the length of a pile, and in fact he always had borings taken.

With regard to the "pile casing forming piers", the ground was simply opened up inside timber piling driven by hand maul at the period of low water of Spring Tides to approximately 4 feet below river-bed-level; thereafter it was cased as stated; the roughening to form a key was necessary so as to form a monolithic mass.

Referring to the canal wall, he agreed generally with Mr. Grunspan's observations. As explained in the Paper, however, the works described were representative in character, and a further reason for the adoption of that type of wall (apart from re-using cut-off lengths of steel sheet-piling) was that the owners of the wharf were financing the cost of the wall in question and did not require any greater depth of water alongside than existed. The conservancy board, however, pointed out that in the future they would require to deepen the canal to 6 feet below mean water-level (note the word "ultimate" in Fig. 5, Plate 1), but they would not agree to contribute the extra cost. In *Fig. 6* it would be noted that the bed of the canal was disclosed in the background much higher than the ultimate dredged level. Hence sometime in the future the Author, or the conservancy board's engineer, might well be accused of "lack of foresight". Needless to say the plans were approved by the authorities concerned, and no settlement of any description had occurred over the years the wall had been constructed.

In the wall illustrated in Fig. 10, Plate 2, the bars shown around the

24-inch by $7\frac{1}{2}$-inch rolled steel joists were nominal, introduced into the beam casing. Mr. Grunspan's further remarks showed that he had studied the Paper very closely and had a clear understanding of that particular design.

The Author wished to record his thanks to all those gentlemen who had taken part in the discussion and for the way in which the Paper had been received.