DISCUSSION
IN AUSTRALIA AND SOUTH AFRICA
ON PAPERS PUBLISHED IN
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Paper No. 5585.

"The Captain Cook Graving Dock, Sydney." †

By JOHN GUTHRIE BROWN, M.I.C.E.

Discussion at a Joint Meeting of the Institution of Engineers, Australia, and the New South Wales Advisory Committee of the Institution of Civil Engineers, held in Sydney, New South Wales, on 30 April, 1947.

After the Meeting had been opened, the Chairman of the Sydney Division of the Institution of Engineers, Australia, invited Mr. T. H. Silk, M.I.C.E., to take the Chair, as representing Mr. A. J. Debenham, M.I.C.E., Chairman of the New South Wales Advisory Committee of The Institution of Civil Engineers, who had been prevented by illness from attending.

A letter from Sir William Halcrow, President I.C.E., was read, and the Paper was introduced in a recorded speech by the Author.

The Chairman, on behalf of the Institution of Civil Engineers; Mr. D. F. J. Harricks on behalf of the Institution of Mechanical Engineers; Mr. V. J. F. Brown, on behalf of the Institution of Electrical Engineers, and Mr. T. H. Upton, on behalf of the Institution of Engineers, Australia, expressed pleasure at the fulfilment of the policy of joint presentation of Papers, and the hope that it would be the forerunner of many such meetings.

Discussion.

Mr. S. T. Farnsworth observed that the Paper had been written by the engineer responsible for the design of the structure, and dealt almost entirely with design matters. Since Mr. Farnsworth had been very intimately connected with most of the constructional work, he looked forward to publication of the Paper on construction matters.

With regard to the dock floor, the Author had stated (p. 298) that:

† J. Instn Civ. Engrs, vol. 28 (1946–47), p. 286 (Oct. 1947).

> "To lead off any pressure water which might accumulate under the floor by percolation or by fissures in the rock, a system of drains was installed under each block. The top of each vertical drain in the centre of the block was sealed by a ball-and-socket relief valve (Fig. 5, Plate 2). These drains have been found to operate satisfactorily."

The dock, as an engineering structure, might be regarded essentially as two parallel gravity dams, of a cross-section of approximately 31 feet at the base and 13 feet 7 inches at the crest. Those small dams were founded upon good-quality sandstone rock and were backed on the outside of the dock by sand filling. The sand fill was necessarily water-charged, and the height of the water-table would be approximately the same as sea-level. Hence, when the dock was filled with water, hydrostatically the dams were in equilibrium, and the maximum stress on the foundations would occur when the dock was empty. The Hawkesbury sandstone on which the dock walls are founded was a good-quality sandstone, laid down in sedimentary strata, separated with bedding planes, which were very often water-bearing. Undoubtedly, therefore, sea-water would percolate the strata underlying the dock floor and generate uplift pressure. Hence, having regard to the generally horizontal bedding planes, it was necessary to prevent excessive uplift pressure forming in planes underneath the floor and above a level where it was not balanced by the weight of the floor and the superincumbent sandstone rock. Thus it was obvious that to core vertical drains through the centre of each concrete floor block would not meet the position. Those drains were therefore extended to a depth of about 30 feet in order to meet the above-mentioned requirement for equilibrium. These sub-floor drains were not shown in Fig. 5.

The installation of ball-and-socket relief valves would appear to be an unnecessary expense. They were designed to permit free relief of water from the foundations into the dock when the latter was empty, but to cut off a reverse flow when the dock was full. It was imperative that the relief drains should operate freely when the dock was empty, in order to relieve uplift pressure. When the dock was full, any reversal of flow would be inappreciable.

It was very probable that, owing to accumulations of silt in the ball valve mechanism, many would rapidly become inoperative unless expensively maintained. Some simple form of trap aperture capable of being quickly and easily cleaned, would be more appropriate.

With regard to the dock walls, the Author had stated (p. 299) that:

> "For construction of the walls, they were divided into units 40 feet long, separated by gaps 5 feet wide."

Mr. Farnsworth was unable to recognize the advantages of that method of construction. The construction of a gravity dam of the very small cross-section involved and subjected to a hydrostatic head of only 45 feet, did not warrant any extraordinary provisions being taken to ensure watertightness between construction units, and the 6-inch cylindrical bitumen seal could certainly be depended upon to ensure watertightness against such a low head. Incidentally, as stated above, any unbalanced hydrostatic head necessarily acted from the fill behind the walls towards the dock when the latter was empty, and actually Mr. Farnsworth believed that any tendency to leakage into the dock was entirely counteracted by bitumen sealing strips placed near to the back face of the wall. Since each dock wall was approximately 1,200 feet, if 45-foot blocks only had been used, there would have been, during construction, a choice of more than sixty block-areas in which to place concrete. With the five electrically driven traversing telfer gantries and subsidiary cranes used to deposit concrete, there could never have been any restriction upon the rate of concrete placement.

It was actually found that the 5-foot gaps greatly increased the cost of construction and greatly lowered the speed at which the work could be carried out. Whether an area to be concreted were large or small, there were certain incidental operations that were common to all, namely, travelling gantries into position; arranging for water services; sand blasting; arranging for registering of supplies of concrete from the transport system, etc. With the 5-foot gaps, approximately 2,100 separate pours of concrete were required. Those pours would have been reduced to approximately 1,100 had the 5-foot gaps been eliminated.

To add to the constructional difficulties and cost, most of the services to the cope, and also the shafts between the two subways, were placed within the 5-foot gaps. From the construction point of view, it should be realized that construction of the formwork for the shafts and the placement of the services were greatly handicapped in such a confined space, resulting in much heavier skilled labour erection costs than would have occurred had those works been constructed in the 45-foot blocks. Moreover, placing concrete satisfactorily in such confined spaces presented great difficulties, with consequent added expense. It was very difficult indeed to use vibrators satisfactorily in the narrow work, whilst cleaning up, scabbling, and sandblasting between lifts was extremely tedious and very expensive.

The use of analogous narrow gaps in the construction of arch dams had been a very useful device for many years past. In that case, however, it was necessary to obtain not only watertightness, but also the transmission of bearing pressures along the length of the wall. Moreover, the ends of the wall were confined by the abutments, which took the arch load. In the case of the long narrow-section gravity dams constituting the dock walls, the problem was not at all analogous, and, indeed, if the 5-foot gaps were allowed to remain open until the 40-foot blocks had undergone approximately their ultimate contraction before the gaps were filled, then it could easily be shown that under the severe range of temperatures experienced in the Sydney climate and the small cross-section of the walls, undoubtedly longitudinal, horizontal shear stresses of a very severe character would have been generated in the walls under extreme summer temperatures. Those stresses would not have been transmitted to an abutment as in the case of an arch dam, and it was probable that rupture would have occurred. The Consultant's representative was advised of those factors before concreting of the walls was commenced, which resulted in the direction that gaps could be filled in between main blocks eight weeks after construction of the adjacent main units.

From the above, Mr. Farnsworth considered that it had to be concluded that the provision of 5-foot gaps for small gravity dam construction was unnecessary and only resulted in extravagant expenditure.

Mr. F. de L. Venables observed that the Metropolitan Water, Sewerage, and Drainage Board had placed the order for the central mixing plant in August 1941. The plant was modelled on that in use at the Grand Coulee dam, U.S.A., and incorporated weight batching-machines for aggregates and cement, and "Koering" tilting conical mixers. That was the first installation of "Koering" mixers in Australia.

It had been obvious to the Board from the first, irrespective of any considerations relating to mass concrete construction, that the proper type of cement to be used in a seawater structure of that nature was low-heat Portland cement manufactured by several Australian cement companies to the Board's specification. The specification was based on that for low-heat cement for the Boulder dam, with certain changes in detail to conform to the Australian standard methods for physical testing. The principal feature in the Board's low-heat cement specification was the provision for control and limitation of the calculated mineralogical composition; in particular the calculated content of tricalcium aluminate was limited to 8 per cent., and in practice the average tricalcium aluminate content was about 4·1 per cent. In all other respects the low-heat cement complied with the Australian Standard Specification for ordinary Portland cement. Extensive tests in the United States had shown that a cement with a content of tricalcium aluminate of that order could be classed as sulphate-resistant, and indicative tests carried out by the Board had confirmed that. Some of those tests had been described elsewhere.[1] A sulphate-resistant cement was advantageous in seawater but, in order to get the highest resistance to both sulphate and magnesium, it was necessary to have also a high quality of surface finish, low absorption, and low permeability; in the case of thin sections, high strength of concrete was also desirable, such as might be obtained with practical mixes with a cement-content of the order of 650 lb. per cubic yard. The Board had arranged for the supplies of low-heat cement for the dock construction in September 1941. There was no difficulty in making that provision, as low-heat cement to the Board's specification had been the principal

[1] *Commonwealth Engr*, vol. 27 (1940), p. 388 (1 June 1940).

type used in nearly all the Board's structures since 1935. The Board had accumulated a mass of information on strength properties, whilst the Australian manufacturers had mastered the technique of production. Low-heat cement was still used in quantity in only two countries, namely the United States and Australia.

The Board had taken up the construction of the dock itself in March 1941, and had organized the plant lay-out to provide for its standard practice in concrete construction, namely, the use of maximum density gradings and controlled water/cement ratio. The Board's concrete inspection system and physical testing laboratory were already available, being a part of the normal organization, and the equipment and methods, including fog-room curing, were incorporated as routine facilities available for the construction of the dock.

When the specification for the concrete work, dated January 1942, was received from the Department of the Interior, it was found that Section 65 stated that the sand and stone grading fractions were to be recombined to provide for concrete of maximum density. The next Clause, No. 66, on the other hand, made it mandatory to adhere to fixed proportions of cement, sand, and coarse aggregates for the various classes of concrete designated by letters and it was impossible to comply with those contradictory instructions. Therefore the matter was settled by fiat. That probably accounted for the table of proportions given in the Paper, in which the size limits of the fine and coarse aggregates were not defined.

The specification for concrete mentioned above also provided for the control of the slump by the water/cement ratio. That could be done

LOW-HEAT CEMENT—MEAN COMPRESSIVE STRENGTHS

Equation of strength : $$S = \frac{14{,}000}{B^{w/c}}$$

where S denotes compressive strength, in lb. per square inch, B the combined class and age parameter, and w/c the ratio of active water to cement by volume, taking 94 lb. of cement as equivalent to 1 cubic foot.

Size of specimen : 6-inch by 12-inch standard cylinders.
Packing : A.S.T.M. Standard.
Curing : Fog-room, between 70° and 75° F.

Age	Nepean Gravel		Crushed Nepean Gravel		Crushed Basalt	
	B	Number of Specimens	B	Number of Specimens	B	Number of Specimens
7 days	9·184	31	—	—	—	—
28 days	5·302	3356	4·533	470	5·882	706
3 months	3·437	651	3·170	69	3·279	70
6 months	2·906	527	2·401	45	2·636	43
1 year	2·706	476	2·361	48	2·769	40

only by departing from the mandatory provisions of Clause 66, in which the proportions of cement, sand, and coarse aggregate were laid down. Therefore water/cement ratio control of the strength was not used. Consistence was the governing factor, and water quantities were altered accordingly.

The table on p. 595, giving values of the principal parameter in the equation for mean strength of concrete made with low heat cement (to the Board's specification), would provide some useful technical information. The Board's tests had shown that the parameter in the equation differed in value for uncrushed Nepean river gravel (mainly quartzite and hard acid and intermediate rock), crushed Nepean gravel, and basalt.

Mr. C. R. Bickford observed that he wished to refer to two features of the Paper relating to design. The first had reference to the specified strengths for concrete. The highest value anticipated at 28 days was 3,100 lb. per square inch, and that was approximately the class of concrete used in the beam and slab work of the fitting-out wharf. The nominal mixture was 1 : 1·2 : 2, and low-heat cement was used throughout. It could reasonably be assumed that the working stress used in design had some relation to the specified strength at 28 days, but it could have been anticipated that, with such a rich mixture, much higher strengths would be obtained, and in actual practice that was the case. The average of a great number of tests showed a strength at 7 days of 4,000 lb. per square inch, and at 28 days a strength of 7,000 lb., or more than double that specified. The value of density and impermeability was fully appreciated, but in establishing the proportions to produce such results, advantage should be taken of the higher strength of the concrete. Mr. Bickford felt that if that had been done, the size of members could have been substantially reduced, and considerable savings in time and money would have resulted.

The second feature was the general design of the fitting-out wharf. Having regard to the fact that the cylinders that were a feature of the construction were wholly dependent on timber piles driven through clay to rock for their support, and the massive type of construction, it would appear that the proportion of dead load on the foundations was out of all proportion to the live load. Mr. Bickford considered that by constructing the wharf as a wholly piled structure, the wharf could have been built much faster and cheaper and with less difficulty with regard to supply of materials.

Another point was that the final resistance to horizontal movement due to pressure from the reclamation at the back of the wharf was provided by the resistance of the clay in front of the cylinder bases, and to some extent by the foundation piles. That necessitated particular care in placing the rock fill under the wharf, and any disturbance of the ground in front of the cylinders might have serious results.

With regard to the keel blocks: although it was decided to use timber so as to conserve steel during the war, Mr. Bickford did not think that at the time the great difficulty that would be experienced in obtaining supplies of timber to enable that work to be carried out was appreciated. The sizes required, the quantity involved, and the high standards that had to be insisted on, made the acquisition of the timber a very serious and, at one stage, a critical problem in preparing the dock for war purposes. However, the difficulties were overcome, and the blocks had functioned very satisfactorily in practice.

The tests that were carried out on the keel blocks were of great interest, as it was the first time that tests in cross compression on large specimens had been carried out. Previous tests had been limited to small specimens. Had the timber been thoroughly seasoned, it was probable that higher results would have been obtained, but circumstances would not permit the necessary time.

The arrangement of the keel blocks in the dock was unusual in providing for such a variety of vessels, from destroyers to the *Queen Mary,* and more than 7,700 blocks had to be prepared.

Mr. James M. Antill observed that the Paper was of very great interest, and its value was enhanced by the additional description and lantern slides presented to the Meeting by Mr. S. S. Harrison, M.I.C.E. Both the Author and Mr. Harrison were to be congratulated, and it was pleasing to know that such an excellent work had been chosen for the inauguration of joint presentations before the two Institutions.

In connexion with the concrete mixtures, it was apparent that a very high class product was obtained, but it also appeared that unnecessarily rich mixtures were used. The Table of 28-day strengths given in the paper (p. 295) showed that in all concretes the specification was greatly exceeded—in most cases more than 100 per cent. excess strength was attained—yet the Author had stated that "high compressive strength was of less importance than density and impermeability with small volume-changes in the concrete". The very rich mixtures would therefore seem to be extravagant; for a dense, watertight, seawater-resistant concrete could readily be produced with $1 : 1\frac{1}{2} : 3$, or even $1 : 2 : 4$, nominal proportions, if the necessary precautions, correct cement type, and proper aggregate gradings were used.

Furthermore, since density, volume-change, and impermeability were paramount, it seemed extraordinary that the specification contained no definite requirements for those important properties. The provision of a low strength-requirement "to allow for the use of low-heat cement" might be justifiable, but density, shrinkage, and other special tests were surely warranted, and specific limits should have been provided for those characteristics. Mr. Antill understood that certain tests were, in fact, made, and it would be of great interest to know the results thereof, as the Paper contained no information on that subject. The concrete workmanship was undoubtedly of a high order, but it would appear that, throughout the work, economies could have been made without sacrificing the excellent quality achieved in the finished structure.

With regard to the dock floor and walls, other speakers had dealt with the question of the 5-foot gaps employed in the wall construction, but it would be of interest to learn why such elaborate precautions were taken to seal all wall joints, whilst in the floor nothing more than keyways was provided.

In order to reduce the risk of surface cracking, certain precautions, explained in the Paper, were taken; particularly the use of smaller panel sizes in the rich 4-inch granolithic concrete for floor topping, and the provision of reinforcement in the faces of all subways enclosed in the dock walls. No mention was made, however, of the result of those precautions, nor whether close inspections were made from time to time to determine their efficacy.

Finally, the Author had stated that "galvanized" plates were used on the docking blocks. Mr. Antill understood, on the contrary, that by far the greater proportion were "galvanited," and he would be grateful for any information concerning the comparative resistance to date of the galvanized plates in comparison with those treated with the new galvanite process.

Finally, he wished to congratulate the Consultants, the Construction Authorities, and the Contractors, for their remarkable achievement in bringing a work of such magnitude to a successful conclusion under war-time conditions.

Mr. W. Hudson observed that the Author had referred to certain features which were incorporated in the design of the dock with a view to accelerating construction operations, including the provision made for building the dock walls in 40-foot units, separated by contraction gaps 5 feet long. Having had experience on the construction of arch dams where that method of construction was used, and also on the Sydney graving dock, Mr. Hudson wished to comment briefly, from the Construction Engineer's viewpoint, upon that method of construction.

At the graving dock, the provision of 5-foot contraction gaps between 40-foot wall units involved one additional vertical joint every 45 feet. That practically doubled the quantity of formwork for the transverse joints—a serious matter on that project, where the rate of progress on concreting depended more on the speed at which formwork could be erected than on any other factor.

In addition to the extra formwork required, the provision of the 5-foot contraction gaps involved the scabbling of one end of each 40-foot unit and the setting of double the number of copper water-seals around the subways where they passed through the vertical joints. During the early stages of concreting, before the gaps had been filled up to dock floor level, they collected water and large quantities of construction debris and rubbish, thus requiring frequent attention.

The Author had referred to the use of timbers spanning the contraction gaps, provided to permit an early start being made on the deposition of sand backfill behind the walls. That could not correctly be claimed as a point in favour of the gap method, as obviously it was a means of overcoming a difficulty which would not arise with the normal contraction joint construction where there would be no spaces into which the sand backfill could rill.

For a work where rapidity of construction was a vital factor, perhaps the greatest disadvantage of the contraction-gap method was the delay which unavoidably occurred after completion of wall units, as concrete might not be placed in the gaps until the initial shrinkage in the adjoining wall units had taken place. In the case of the graving dock, the length of the waiting period varied, but in general 8 weeks was stipulated.

Mr. Hudson admitted that, notwithstanding increased construction difficulties, delay, and greater cost, there was justification for the adoption of the contraction-gap method of construction for structures relying on arch action for their stability, as in such cases the avoidance of shrinkage effects at transverse joints was of paramount importance; but that argument did not hold for the gravity walls of the dock.

With regard to the Author's remarks on the watertightness of the dock walls achieved by the adoption of the contraction-gap method, experience had proved that, with water-stops of modern design, there was no difficulty in effectively sealing contraction joints of the type normally used in gravity walls.

To supplement the Author's observations on pressure grouting, Mr. Hudson stated that, in addition to the grouting across the dock entrance and near the main caisson groove, as provided for in the specification, a grout curtain was also placed completely around the periphery of the dock. During the early stages of the work, limitations were placed on injection pressures, owing to the fear of lifting rock beds, and little success was obtained. Later, however, permission was granted to use higher pressures, immediately resulting in satisfactory grout penetration of the rock joints and seams. By the adoption of the stage grouting method, the possibility of lifting rock beds was eliminated.

Engineer-Captain R. Berry-Smith R.A.N., M.V.O., observed that he realized that, in many cases, comparisons were odious, but he felt that it would be of interest to compare the Captain Cook dock with another large dock which was under construction in Australia at the same time and with which he was closely associated.

For some years, the question of the construction of a large graving dock at Cairncross, about 4 miles below the city on the southern side of the Brisbane river, had been under discussion by the Queensland State Government, and it was agreed in 1942 that the Commonwealth should subsidize the State so that the latter should build a dock which was finally fixed to the following dimensions:

Length 800 feet to the inner sill; 850 feet to the outer sill.
Width 110 feet inside the bilge altar.
Depth 30 feet over the sill at low water of ordinary spring tides.

That dock was considered to be capable of accommodating the largest ship ever likely to use the Brisbane river and had now been approved for warships up to light fleet carriers. It was suitable for merchant vessels up to about 30,000 tons, that was, about the size of the new *Mauretania*.

The site fixed for the dock happened to be a sandstone knoll, the only one for miles, and adjacent to the Hamilton swinging basin.

The sandstone was of a friable nature, able to stand up under its own weight and easy to shatter with light charges into sizes capable of being handled by mechanical shovels and motor-trucks. Borings indicated few faults and subsequent experience proved it to be practically impervious to seepage from the river.

The main excavation and all the dock construction work were carried out in the dry, with the river held back by a wall of the original sandstone capped by a few feet of temporary work to retain the tides. Subsequently all that was required to bring the dock into use was to blow away the retaining wall and dredge the entrance channel.

The caisson was generally of a similar type to that fitted in the Captain Cook dock, but without the ballast tank, and was fitted with electrically-driven pumps. The caisson was constructed inside the dock and was floated up with the first entry of water. A steam-driven pumping plant, already available locally, was installed for the dock.

Keel blocks and bilge blocks were built of red hard woods from Northern New South Wales and Southern Queensland, except for caps, which were of Queensland hoop pine, a semi-hardwood by European standards. Therefore it had been possible to dock both merchant ships and warships without changing the caps.

A long wooden breast wharf was constructed in line with the dock cope away from the river, and the dock was equipped with high-pressure fire services, gravity circulating main, fresh water, low-pressure air, alternating-current and direct-current terminal boxes, and a 40-ton stiff-legged crane in addition to the small dockside cranes for light work.

Actually the whole project was constructed at high speed under the shelter of the highest Allied Works Council priority and completion was largely governed by the availability of materials, which were kept to the minimum and turned out to be practically 100 per cent. Australian.

That matter had been brought forward simply to show how circumstances and location could govern the means whereby a certain end was to be achieved.

The designers of the Captain Cook dock had been faced with what appeared to have been some appalling problems, only able to be solved by very expensive methods; whereas, with the Cairncross dock, Nature had provided what turned out to be practically an ideal site; all that man had to do was to make the best use of it.

The Author, in a written reply, expressed his pleasure at the interest with which the Paper had been received, judging by the remarks of the various speakers and the many valuable points for consideration which they had raised.

Mr. Farnsworth had brought up certain construction matters of interest. The Author could not agree, however, with his conclusion that the ball-and-socket relief valves were an unnecessary expense. They provided a very useful safeguard against any large volume of water under pressure passing under the dock floor. With a scheme of the magnitude of the dock, it was to be expected that some divergence of views on matters of constructional detail would arise as between the Consultants and the Commonwealth Engineers responsible for the construction work. Those differences were all amicably resolved during the course of the work, but one of the major items had been Mr. Farnsworth's proposal to eliminate the 5-foot construction gaps on the dock walls. That matter had also been referred to by Mr. Hudson. Both speakers had set out in considerable detail their criticisms of the 5-foot gaps, but it would be appreciated that an equally large list could be prepared dealing with the points in favour of those openings. It was not proposed by the Author, however, at that late stage to go into the matter in detail, except to say that the pros and cons were fully examined by the Consultants and that the Admiralty were also asked to express their views. As a result, it was unanimously decided that the benefits, both during construction and afterwards, from the provision of those gaps definitely outweighed any disadvantages. The exceptionally satisfactory condition of the dock walls shown since the scheme was completed—especially the absence of cracks—had reinforced the correctness of that decision.

Mr. Antill's conclusion that a 1 : 2 : 4 concrete was satisfactorily resistant to the effect of sea water was certainly not borne out in the Author's experience, especially with reinforced concrete.

Both Mr. Bickford and Mr. Antill had referred to the actual compression results on concrete cylinders being so much greater than the results specified by the Consultants, and had deduced that mixtures with less cement could have been adopted. The comparatively low compressive strengths specified by the Consultants were arranged deliberately. It was expected that results ranging from 50 per cent. to 100 per cent. higher would be obtained in actual practice and that only some serious defect in the cement, sand, or aggregate would give results below the specification. The purpose behind that was to allow the proportioning of the concrete mixes to be given as wide a latitude as possible, so as primarily to obtain a dense, impermeable, and easily workable concrete completely resistant to the action of seawater without, at the same time, necessitating undue attention to obtaining high compressive results, which were much less important than the other factors. For the structures in contact with sea water comprising the dock and its ancillary works, the Author was satisfied that the long life necessary for them could be assured only by an ample quantity of cement in the concrete coupled with proper aggregate grading and maximum density. The warm waters of Sydney Harbour were notoriously destructive to maritime works, whether of timber, steel, cast iron, or concrete, and that fact was kept prominently in view throughout the design of all parts of the project.

Mr. Bickford had referred to the possibility of constructing the fitting-out wharf as a completely piled structure in place of the cylinder design chosen. That would have necessitated piles from 80 feet to 110 feet long to reach a foundation capable of carrying the very heavy loads and would have presented serious difficulties in construction.

Mr. Antill had asked why the gaps in the dock walls were elaborately sealed while the dock floor was provided with keyways only. The dock walls were subject at all times to heavy water-pressure on account of the waterlogged filling behind the walls, leakage from which into the subways would be very objectionable and harmful to the equipment. The dock floor, on the other hand, was not subject to continuous water-pressure, and that was restricted to the minimum by the relief openings.

Paper No. 5587.

"The Sturrock Graving Dock, Cape Town." †

By David Eric Paterson, O.B.E., M.C., B.Sc.(Eng.), M.I.C.E.

Discussion at a Meeting of the South African Institution of Civil Engineers held in Cape Town on 11 June, 1947.

Discussion.

Mr. B. W. Wilson observed that the Author had referred to the phenomenon of range action in Table Bay harbour, but had remarked that the full implications of that action, in its effects on the Sturrock dock, had not been ascertained at the time of writing of his Paper. Through the courtesy of the Chief Civil Engineer, however, it had been possible to show to the Meeting movie-photographs since secured, which illustrated some of the features of range action, and, more particularly, its effects upon the

† J. Instn Civ. Engrs, vol. 28 (1946–47), p. 328 (Oct. 1947).