Engineering works at Tilbury Docks, 1963-67

R. F. J. SMEARDON, E. NEWTON & F. A. PAGE

Mr A. H. Beckett, Sir Bruce White, Wolfe Barry & Partners

Everyone will have noticed, how with every successive British dock strike, more and more entrepôt traffic has been diverted to newly improved continental ports at London's expense. It is encouraging to see the really bold effort that has been made at Tilbury to recover lost ground. It may not be too late to get back some of the two million tons per annum of general cargo that has been lost over the past ten years.

198. One must commend the Port of London Authority (PLA) for its modern attitude to port planning, giving large land areas to each berth and keeping the rail tracks away from the quay. This has attracted operators of packaged timber and container ships. In answer to criticism of what might appear an extravagant provision of land, in our experience on No. 39 berth now being fitted for private use container ship duties, the thirteen acres at the berth plus five at the annex are proving barely sufficient to accommodate the container handling equipment, even though this is based on stacking containers five high. Very few ports can offer sufficient land near a deep water quay, and it was because the PLA could provide this facility with good road and rail access that they have secured this new form of high capacity traffic.

199. I should like to ask some questions. First, container ships turn round in about two days whereas the traditional general cargo carrier takes about three weeks, so that for each berth there is potentially ten times the number of ship arrivals and departures. When Tilbury Dock was half its present size it had two lock entrances, now it has only one. With the ability to take as many as ten times as many ship movements, do the Authors see any need for special measures in ship handling through the lock or even the future construction of another lock? If so, where will it go?

200. Second, it is mentioned that the dredging spoil was dumped in Black Deep. In 1957 Sir Claude Inglis and F. H. Allen gave a Paper showing how radioactive trace elements revealed that spoil dumped in this Deep comes back up the river. Has this natural phenomenon now ceased?

201. Third, the Authors describe the convenient method of bringing to site material for reclamation dredged in Lower Hope Reach and dumping it on the river bed near Tilbury, whence it is redredged and pumped ashore. I have seen the hopper dredgers engaged on this exercise. They are driven with considerable verve and use the tide to assist their passage. They make their way downstream on the last of the ebb, and may be seen returning on the first of the flood. The dumping off Tilbury is characterized by an area of river even more discoloured than normal moving happily upstream. I should be interested to know if the mud reaches upstream are now losing depth at a rate greater than normal.

202. Fourth, the Authors state in reference to the sinking of monoliths that the permissible mispositioning of a monolith is 12 in. I should like to know their intended measures if this were exceeded, would the monolith be destroyed and rebuilt and what is the significance of filling the monoliths with concrete at night rather than in the day time?

Mr P. A. Banks, John Taylor & Sons

In § 113 mention is made of soil and surface water drains including three pumping stations: my firm was asked to design the drains for Stages II and III. The problems

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were two-fold. The first is common to all areas of reclamation on this sort of alluvium, that is, the excessive settlements which occur. It has to be borne in mind that the ground level is being raised some 15 ft by hydraulic fill and settlements of the underlying alluvium are therefore substantial.

204. The second problem, which to most of the design work at Tilbury did not represent a problem at all, is that the site is virtually flat. It is convenient, of course, for roads and railways, but not for drains.

205. As far as the foul sewer system is concerned, we overcame both problems by using three pumping stations and relatively steep gradients to the sewers, and we feel that any settlements which occur will be accommodated by these two features. The surface water system presented more difficulty. The fall that there is on the site is away from the dock, so only a small part of the impermeable area could be drained directly into the docks. Pumping into the docks was considered but rejected since a natural drainage channel existed at a level which would provide gravity drainage to the Thames at low water. The sewers run the whole length of the dock, which is a considerable distance, and are of 5 ft or more in diameter at the bottom end and 16 ft deep. This resulted in considerable problems so far as settlement was concerned even when the sewers were being laid. It was expected that the sewers, which were flexibly jointed thick-walled reinforced concrete pipes, would be laid on a granular bed, but the bed had to be changed to concrete with 2½ in. joints corresponding with the pipe joints.

206. The surface water sewers were obviously the greatest problem from the point of view of settlement and piling was considered. It was rejected on account of cost which would have been in the region of £150 000. It is considered that despite settlement there will be sufficient hydraulic gradient overall to discharge the flows which will be running off these very large impermeable areas and since many of the sewers are large enough to enter and are situated in service reserves the future maintenance task will be acceptable.

207. The foul sewers were originally to have been flexible jointed rigid pipes, but these were changed on the suggestion of the contractor to pvc pipes to achieve a fully flexible pipeline which would accommodate ground movement better.

Mr J. T. Williams, National Ports Council

In § 5 of the Paper the sentence 'In the event, the (Rochdale) report fully supported the proposals for new berths at Tilbury'. As far as Stages II and III of this development were concerned, these were covered by the Harbours Act 1964, which required approval by the Ministry of Transport on the advice of the National Ports Council, and this development itself pays a great tribute to the speed at which the work has taken place, the planning and foresight. I think one should mention that the National Ports Council and the Ministry of Transport were also fully aware of the importance of this kind of development if we were to have the chance of competing with continental ports.

209. It is interesting that in the Interim Plan of the National Ports Council some ten berths of 650 ft in length were recommended at Tilbury, but in fact the number of berths in this development is only eight, although they are considerably longer. The total quay length, incidentally, comes out at about the same or slightly more than that recommended in the Interim Plan. This illustrates the rate of change and the difficulty one is faced with in making appraisals and assessments of such schemes.

210. Referring to § 70 dealing with the roll-on, roll-off berths, mention is made of the requirements for supporting the stern ramps of vessels. There has been some effort to produce standardization in the realm of roll-on, roll-off ships, and it is interesting that in Fig. 22 the vessel shown is a bow loader and not a stern loader. When the berths were designed only stern loaders were in mind. Have any adjustments been necessary in design, or have any difficulties been experienced with bow loaders?

211. Dealing with roll-on and roll-off berths, I think the berths are shallower than the remainder of the dock, and the type of construction provides for obsolescence in that they can be fairly easily removed at reasonable expense.

212. I welcome the load tests referred to in Appendix 4, and hope that the work can be extended to provide more reliable information about the loads actually imposed on dock structures in practice.

213. Figure 1 should read 'below Trinity high water' (below THW), not 'above THW'.

Mr D. G. McGarey, Chief Engineer, British Transport Docks Board

This instructive Paper describes an important and very interesting dock development, the total cost of which is indicated as being of the order of £20 m. Although larger in this respect than the developments at King George Dock, Hull, carried out four years earlier,⁹ there are several points of resemblance. The main differences are that at Hull new quays were of piled and deck construction over existing pitched slopes, whereas at Tilbury they were almost entirely of monolith construction in a 'green-field' site; and that the four years' interval brought in the need to provide for roll-on, roll-off services and to change over from conventional shed berths to open container berths.

215. In both cases, however, the works described comprise stages towards the maximum exploitation of an existing impounded dock system, situated on a tidal river many miles from the open sea, without any attempt to enlarge the basic scale of the system by constructing a new lock entrance or deepening the approach channels by dredging. This parallel is indeed emphasized by the construction at King George Dock, Hull, within the past four years, of two roll-on, roll-off terminals and the major extension currently in hand whereby 4750 ft of new quays, including container berths, are being provided in a new south-east arm.

216. The decision to use monoliths for the new quay walls appears to have been based on previous experience, and the Paper gives little indication that alternative methods of construction were considered. It is, however, interesting to note from § 88 that the crosswall at the north end is being constructed in steel sheet piling, and it would be of value to know the reasons for making this change. In the Western Docks extension now in hand at Southampton it was originally intended to use monoliths, following the precedent of the 7000 ft quay of the West Docks constructed nearly forty years ago. Close investigation of the site conditions and modern techniques showed, however, that an anchored steel sheet piled wall, with reinforced concrete relieving platform, was practicable and offered a substantial saving in cost. If similar considerations have now been found valid at Tilbury, it is possible that the cost of the quay walls might have been reduced quite materially.

217. The trial of pressure cells was, I am sure, well worth while even though the results may be thought disappointing. Have the Authors any suggestions to make for future trials of this kind—preferably at less expense?

218. It would also be interesting to know, in connexion with the monolith work, what order of settlement, if any, has been experienced since 1964/65, and what the effects on costs were of accelerating the rate of construction as described in § 89.

219. The 206 ft spacing of wall ladders may prove to be too great if the new dock regulations are approved as now drafted. Fortunately, it may easily be possible, with the monolith construction, to reduce this spacing to 103 ft.

220. The transit sheds Nos 36 and 38 offer an instructive comparison with those of very similar size at Hull four years earlier. One is grateful for the breakdown of costs given in Appendix 3, and must assume that a direct comparison with those of Paper 6752, Appendix 2, would not be misleading. On this assumption the respective costs are 49s/sq. ft at Tilbury and 36s 3d at Hull, an increase of 35%. Even if a general rise in prices of, say, 5% per annum has occurred, the comparison seems

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clearly to be in favour of Hull, in spite of the all brick wall construction which has received some criticism on the score of being unnecessarily expensive.

221. Nevertheless, the Tilbury design appears to be a good one, and might well have shown greater economy if it had been possible to pursue the original idea of constructing a larger number of sheds and so spreading over more widely the cost of steel moulds for the concrete units.

222. The use of Stelcon rafts for floor surfacing is no doubt due to a desire to cope with settlement problems. They are very expensive, however, and it might have been worthwhile to use an asphalt wearing surface on a lean mix concrete base. Any inequalities resulting from settlement could readily be made good by local relaying of the asphalt.

223. The open battery charging shed is a simple and cheap means of providing the service. If covered accommodation for the trucks is not required, it is more economical than the enclosed shed constructed at King George Dock, Hull. At Southampton, however, a change over from battery power to liquefied petroleum gas fuel is being made. A considerable saving of charging time is made possible by the rapid change over of gas cylinders.

224. As for the roll-on, roll-off berths, being able to omit any permanent bridge ramps, relying only on the ships' ramps for overcoming differences of level, has obviously made possible a considerable saving of cost. What is perhaps more debatable is the omission of any form of stern fendering. What provision is made for safeguarding the vulnerable parts of the ship and jetty against damage by accidental impact? It would seem from Fig. 19 that the position of the ship in relation to the fixed structure is not subject to much tolerance, and it would be useful to know if reliance is placed entirely on the ships' ropes for this purpose.

225. It is noted from § 75 that the jetty was designed for HA loadings and checked for 35 units of HB loadings. In B.S. 153: Part 3A, 1954, reference is made to 45 units, and this standard was recommended in § 31 of the PIANC report on roll-on, roll-off ships and terminals published in 1966.

226. Finally, I should like to comment on the new impounding station, the design of which has obviously received a great deal of thought. Here again, there is a similarity with the rather smaller installation at King George Dock Hull, where three pumps each of 170 cusec at a design head of 11.2 ft are in service. Experience with operation of the axial flow pumps at Hull suggests that the limitation of head imposed at Tilbury by silt-content considerations is perhaps fortunate, and the decision to employ variable pitch impellers with constant load control may have been taken in the light of the same manufacturer's experience with fixed blade impellers at Hull. In all the circumstances I am sure that the care taken at Tilbury to ensure the best possible flow conditions at the pump suction intakes is entirely sound, as also the decision to use the balanced diaphragm type of anti-siphon valve on the discharge side of the pumps.

227. There can be no doubt about the importance of undertaking a comprehensive survey of silt content variations in the river water in cases of this kind. In the present instance the results have shown the need for rather elaborate intake works. As compared with conditions at Hull a very noticeable difference is to be seen in the particle size distribution. There the majority of particles fall within the larger categories of silt. At Tilbury the smaller silt and clay fractions preponderate. At the end of § 142, the clay particles should be shown as 'below 0.002', not 0.006.

228. The value of these results, however, must depend on the validity of the sampling technique and here I feel that the method described in §§ 136 and 137 leaves much to be desired. For nearly nine years silt concentrations on the Humber have been continuously measured and recorded at a number of sites by means of automatic recording siltmeters, devised and installed by the British Transport Docks Board's research department. The constantly fluctuating indications show clearly that no system of intermittent sampling can possibly yield reliable results.

Mr J. R. O'Donnell, Port of London Authority

The greater ease experienced in the sinking of monoliths on the recent contract when compared to the difficulties recorded from contracts in the mid-fifties, may be attributed to the change in design of the shoe rather than to the use of bentonite. Experience gained on Stages II and III indicates that the 2 in. lip on the outside face of the shoe, the reduction of the cutting edge width from the 9 in. and 6 in. as mentioned in the Paper, to 4 in. and 3 in. and the raising of the internal crosswalls, were more significant factors in the sinking of monoliths. This was clearly demonstrated by the fact that the last two monoliths in Stage III were sunk successfully without the use of bentonite at all. In fairness it must be added that the top 8–10 ft of material through which these two monoliths were sunk was pumped ashore ballast which offered less resistance to sinking than the natural clay.

230. Mr Beckett raised the question of drifting of monoliths beyond the 12 in. tolerance allowed in the design. Drifting only became significant when monoliths moved forward into the dock to come outside the cope beam. Typical results of such movement in Stage II sinking are given in the Paper and only on the West Side of Stage III did monoliths drift beyond the 12 in. allowance and then only to a minor degree near the dock bottom. When dredging of Stage III is complete, cutting away the lower faces above dredged depth will be undertaken on the few monoliths requiring such treatment. This partial incorrect positioning arises essentially from a lack of plumb rather than a drift forward. If monoliths move backwards away from the dock then the cope beam cantilevers a little further. Restrictions imposed by adjacent monoliths on both side faces tend to prevent movements sideways.

231. Referring to the anchored bulkhead, I should like to fill in some detail and possibly answer Mr McGarey who spoke about the switch in design from monoliths to an anchored bulkhead. There were two reasons. As construction of Stage III followed so rapidly, considerable quantities of steel sheet piling from Stage II works became surplus and economies could be effected if this material was incorporated in the design for Stage III. The second reason is that the ballast stratum at the head of Stage III is generally higher over the width of the dock making an anchored bulkhead a more viable proposition. Surplus Frodingham No. 5 section piles were lengthened and Frodingham section No. 3N were strengthened by means of a 24 in. × 9 in. universal beam welded to the rear face. These piles are currently being driven and in fact the magnitude of the job is such that it almost warrants a separate paper.

Mr I. S. Greeves, Director, Mowlem (Civil Engineering), Ltd

Regarding piling at Tilbury the problem has always been to decide whether the toe of the pile will gain sufficient bearing capacity in the gravel, or whether it will penetrate into the chalk. Where the gravel layers are thin it is likely that the pile will penetrate about 17 ft into the chalk before a satisfactory set is obtained.

233. For many years the PLA used 17 in. octagonal piles usually about 75 ft long. This meant heavy pile handling equipment and 90 ft pile frames. About 12 years ago the Authority experimented with the use of prestressed piles with blunt ends. These usually obtained a satisfactory set about 3 ft into the gravel.

234. In 1963 the Mowlem Cobi pile was introduced. This pile consisted of a thin corrugated tube fitted with a flat steel shoe. The tube was given rigidity by a pneumatically expanded mandrel which fitted exactly to the corrugations.

235. The pile was then driven, the mandrel extracted and reinforcement and concrete placed in a similar manner to other driven in situ type piles.

236. The difficulties with this pile were that leaders over twice the length of the casing were needed to insert the mandrel into it. The length of pile is limited to about 50 ft due to the fact that Ministry of Transport regulations are such that it is uneconomical to move very large machines about the country. These difficulties

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do not arise in the United States where the pile was first developed. Undoubtedly the flat ended pile was ideal in the ground conditions encountered at Tilbury and when the BSP cased pile was developed the PLA decided to use it in the construction of the 35/37 Berths to be used for roll-on, roll-off traffic. As the pile is driven with an internal drop hammer much lighter plant can be used than for the Cobi pile and extensions can easily be carried out by welding additional lengths of tube on to that already driven. The piles on the approach were driven by a 38 RB excavator and those over water were driven using a floating pile frame.

237. One of the difficulties encountered in building the jetty was the release of the soffit shuttering. Usually jetties are built in a tidal river or estuary and the release of the soffit shuttering can be accomplished on the ebb tide. However, in building the jetty in impounded water we found that there was only 3 ft between the water and the soffit of the deck. A system was developed whereby yokes were supported from the top of the piles with a cross-head and the shutters hung from them. By this means it was possible to release the soffit shuttering sufficiently to allow it to be withdrawn by means of a crane standing on the completed deck. By careful planning of the sequence in concreting operations of the deck it was possible to remove all the shutters fairly easily.

Mr J. R. F. Stephens, John Howard & Co. Ltd

Much attention was given to the detailed planning of the original contract for Stage I consisting of 40 monoliths and it is interesting to note that none of the basic concepts had to be altered for the 231 monoliths eventually completed.

239. The monolith shutters were designed so that the complete outside shutter, with spreader, could be lifted by a 10 ton derrick. This determined the lift height of 9 ft. The core shutters weighed 4 tons each and the whole assembly comprising 270 sq. yd of shuttering was held together by 24 H.T. steel ties and could be stripped, oiled and refixed in half a day.

240. Concrete was transported in special bodied lorries and placed with 2 cu. yd skips except for underwater concrete where 4 cu. yd skips were used. On the question of concreting at night raised earlier in the discussion by Mr Beckett, this was purely a matter of programming and obtained maximum use from expensive plant. Monolith walls were shuttered and concreted by day, underwater sealing which required a special mix and some 10 hours' continuous work could be fitted in better at night. Hearting was also generally done by night. Both these operations required few men. The mix for hearting was reduced from 10:1 to 20:1 after cracking was found in the 3 ft monolith walls. At the peak period in Stage II some 7000 cu. yd of concrete per week was being poured.

241. The monoliths were sunk by grabbing with crawler excavators with 1 cu. yd grabs; this proved much quicker than grabbing with derricks but the biggest single saving in time over previous work at Tilbury was our ability to sink monoliths without kentledge. This was due to our redesign of the cutting edge and bentonite lubrication, both of which have already been described.

242. When the effect of these measures could still only be estimated, my company contracted to sink the first 40 monoliths using only 750 tons of kentledge instead of the 2000 tons allowed in the tender. At the start in Stage I 100 tons was used on the first lift to stabilize the monolith, which later proved unnecessary, and 250 tons was used for sinking into the ballast. In Stage II no kentledge at all was used for sinking the monoliths but the Port of London Authority still required 250 tons of kentledge to be placed after sinking was complete by way of a test. In some cases where the monoliths were founded near the chalk a 500 ton test was applied. It is difficult to assess how much the ease of sinking can be attributed to bentonite lubrication. It was used very thoroughly at the start but as work progressed it was observed that the omission of bentonite on occasions did not seem to have any marked effect and the last two

monoliths were sunk without bentonite early this year. However, bentonite was used to sink the 47 ft dia. monolith at the Impounding Station which was considerably lighter per sq. ft of surface area, again without the use of kentledge. Had the same amount of kentledge been used for each of the 1550 lifts of concrete in 7 ton blocks, as on previous monoliths at Tilbury it would have required over half a million crane lifts.

Mr D. H. Little, Walter C. Andrews & Partners

In the past eighteen months I have been to Tilbury many times and I have seen a great deal of the work, and can do no more than say these monoliths at £600/lin. ft were very cheap indeed. I would add that they were very cheap—for monoliths! Some monoliths at Rosyth and Singapore were 50 ft square and 130 ft deep, and pro rata today would have cost nearly £3000/lin. ft, which is not cheap for a wharf wall. But the ground conditions there are so bad that I doubt whether a wall could be built in any other way.

244. Because the ground conditions were so good at Tilbury, a sheet pile wharf would have been adequate and satisfactory, at about two-thirds of the price. Against that, the wharf is not the full cost of the project. There are dredging and other factors to be taken into account, and there is in this extra one-third a solid face of mass concrete compared with a face of $\frac{1}{2}$ in. steel.

245. As far as ground pressures are concerned, water is the most significant factor. Coarse and fine aggregates by themselves stand up well, but as soon as cement and water are added, the mixture for a short time behaves as a complete fluid. Ballast can be dug in a pit and will stand up 20 ft vertically until the water table is reached; then the digger moving over the wet ballast will convert it almost to a fluid. Even a good clay with a 4000 lb/sq. ft shearing will split off like slate if water gets into the fissures. This variation of behaviour with water content is such that perhaps a single quantitative solution is not possible. With maritime works there is the advantage that water is present all the time. In the case of a dry dock, when it is emptied there is saturated fill behind the walls, and for a tidal berth the net pressure can be quite low if the water can be balanced at low water. With a non-tidal berth, the pressures behind and in front are almost completely balanced and the wall has really little to do.

Mr D. J. Bright, Ports Division, Ministry of Transport

Unlike Mr McGarey I have an interest in the Port of London Authority. Since the Harbours Act this development can, of course, be looked upon as one of the Minister's investments, and it has been my great pleasure to be able to go and look at the works when they were in progress.

247. My first point concerns § 112. When we became aware in 1965 that there would be a great volume of traffic resulting from this development, we had to do something about the road access which up till November 1967 was via what was known as Hairpin Bridge, a very bad access over the railway which cuts off the port from the rest of the land.

248. There has been a scheme for some years for a re-routing of the trunk road which runs over the bridge, and in fact some piers were erected on each side of the railway line, in the position of the line shown in Fig. 1. Unfortunately the development was not proceeded with then, and it was not until 1965 that we really had to get things moving on that site or have been in trouble, particularly with container traffic. It was then a question of fitting the scheme into the road programme, which requires 31 steps according to Colonel Lovell (County Surveyor, West Riding of Yorkshire). Some of those steps had been taken, but a number had to be taken after that. In the end part of this went in the programme, and we managed to include stages one and two of the road, which included the bridge over the railway.

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249. They were completed in substance at the end of November 1967. The extension to the A13 will be delayed. We managed to get over the railway in 1967, but because there are 200 objections plus an objection by the Urban District Council, I do not think we shall have the rest until 1969, which is a great pity.

250. The roll-on, roll-off jetty is stated to have been designed for HA loadings and checked for 35 units of HB loading. As the access bridge to the docks is designed for 300 ton loads, what will happen to abnormal loads?

251. To take up a point made by Mr McGarey: the cost of similar quays at Rotterdam, according to an earlier paper, appeared to be about £315/ft as against £543/ft. It seems to be a similar case, and I wonder whether it was a question of greater convenience and speed to have this design as opposed to any other. It would also be interesting to know the cost of the crosswall berth.

Mr R. H. Walpole, Peter Reid, Walpole & Partners

My firm was appointed early in 1966 to develop berth 42 for the importation of packaged timber and paper and pulp. We started work on site in November 1966 and finished by about the end of May 1967.

253. Figure 37, from a photograph taken at about the end of May 1967, shows the berth more or less complete. The only work not finished is a strip of paving at the north edge. During the construction period we laid approximately 18 acres of reinforced concrete paving, erected a shed of about 2½ acres in extent, and built a small office block.

254. The Paper covers a very wide range of work, and I should like to comment on

Fig. 37. Berth 42, Tilbury ; end of May 1967

a few points which affected our work, or in respect of which we did similar work to that covered by the Paper.

255. I think the settlements which are expected at Tilbury present the major engineering problem. In the Paper the cohesion of the alluvium is given as 300–600 lb/sq. ft. For our boreholes we obtained values varying between 150 and 1300 lb/sq. ft, and it can be seen that this alluvial material is very soft and rather variable. We estimate that the settlements which will take place over the loaded areas of our site will be between 12 and 18 in. over 10 years. This is rather more than the figure given in the Paper of $\frac{1}{2}$ in./year. In support of our figures I would say that our shed floor has already sunk between 2 and 3 in. in six months. It is loaded with reels of paper.

256. The costs of sheds are given in the Paper as between 47s and 49s/sq. ft. Luckily our client's requirements were simple: we were able to omit sides, ends and doors, and as a result we achieved a shed cost of 17s 1d/sq. ft. This figure is made up of foundations 3s 8d, steel work and sheeting and gutters 10s 1d and paving 3s 4d. Our shed was a fairly simple frame construction. The foundations were piled, and we have arranged that the floor slab will slide down the sides of the pile caps and not break its back upon them.

257. Over 100 000 tons of cargo have already been unloaded since July, and from that point of view our client is delighted.

Mr M. A. R. Davis, Mowlem (Civil Engineering), Ltd

Considerable resources were employed initially on site to clear away the existing bushes and scrub over the area of the new works, including the drainage and filling of various ponds and dikes. Imported fill was used to form embankments to carry the new perimeter road and railway lines and progress attained to enable these new facilities to be opened on the programmed date. Extensive areas have been reclaimed by using material pumped ashore.

259. When Stage III work was begun further embankments were constructed and again progress was such that the facilities were opened as programmed.

260. Work is now in progress on the installation of drainage and general services to the new berths.

Mr E. T. Haws, Soil Mechanics Ltd

I should like to congratulate the Authors on a very useful and informative Paper. The overall planning aspect of concentrating large scale development at Tilbury seems an essential step in rationalization of the transport system and it is to be hoped that we are in time to compete, where necessary, with Europoort.

262. We have worked with the Authors on certain specialist aspects of the development project. Referring to § 22, I note that the Authors select a shear strength in the alluvium of 300 lb/sq. ft for the top 24 ft followed by a uniform increase thereafter to 600 lb/sq. ft at a depth of 49 ft. In connexion with stability performance in reclamation areas No. 2 and 3 we carried out a number of field vane tests and laboratory quick undrained shear tests in 1966 and 1967. The results are shown in Figs 38 and 39.

263. In general, a more realistic estimate of the in situ shear strengths of these soft alluvial deposits can be found by field vane testing as it is almost impossible within economic limits to avoid disturbance during sampling and laboratory testing. The figures show the high degree of variability of shear strength in the recent alluvium but for the analyses then in hand it was felt justified to use somewhat higher shear strengths for the upper layers of the alluvium compared with those adopted by the Authors. The natural marshes in these areas have a somewhat stronger crust, which can be explained as a result of desiccation consolidation.

264. Laboratory quick undrained triaxial results are noted opposite the actual

Fig. 39. Shear strength plotted against depth

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Fig. 40. Soil profiles for settlement analysis

depth of sampling and in general the values for the soft materials are lower than the equivalent values obtained by field vane tests.

265. Referring to Fig. 6 and Appendix 4, the behaviour of cell No. 46 in giving a pressure apparently greater than the equivalent hydrostatic value is indeed odd. However, our work at the site indicated considerable variability in the in situ densities of the alluvium and values between 97 and 105 lb/cu. ft were adopted for calculations. Thus, the broken line on Fig. 6 indicating pressures calculated with no shear strength in the alluvium, could possibly be moved further to the right if some higher densities were in fact encountered in that area. Also, the full line of Fig. 6 is noted as having been calculated allowing for laboratory shear values. These could be lower than the undisturbed values and thus the full line could possibly be moved somewhat to the left. The pressure cell readings might thus be encompassed within a suggested new range between these two lines. An additional effect is the probable local and temporary reduction in shear strength due to re-moulding at the time of sinking the caissons but our work on another similar site shows that the shear strength is likely to return to approximately 80% of the undisturbed value within a month of the end of the disturbance.

266. Referring to § 41, were any special measures taken with the steel shutters when concreting was taking place at low ambient temperatures?

267. Referring to Fig. 19, what allowance was made for negative skin friction in the design of the BSP cased piles?

268. Referring to settlement figures given in §§ 100 and 107, and to the order of settlement of $\frac{1}{2}$ in./year referred to in § 108, Figs 40 and 41 show a settlement prediction for reclamation areas Nos 2 and 3, based on laboratory oedometer tests for compression index and coefficient of consolidation. Fig. 40 shows three different typical soil profiles assumed in the calculations, and the reclamation fill overlies approximately 15 ft of alluvium followed by a peat band, and then a further band of alluvium similar to the top one; in two cases there is a second peat band on top of the foundation gravel and in one case there is no peat at all. The settlement time prediction is shown in Fig. 41 and for the most compressible case with the peat bands of maximum thickness a total of 5½ ft of settlement is predicted with 75% consolidation in 46 years. It appears that both quantity and rate of settlement will remain appreciable for a considerable period. These curves are based on constant groundwater level equal to the

Fig. 41. Rates of settlement from laboratory tests

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natural value and the total settlement would be reduced by between 30 and 80%, depending on the case, were the groundwater level in the reclamation fill to rise to dock impounded level.

269. Piezometers have been installed to check the rate of settlement. It is early yet to say definitely what effect the readings will have on laboratory predictions, but there is an indication that the field c_v may be approximately 25 sq. ft/year compared with 4 sq. ft/year adopted from the laboratory oedometer tests. Thus the actual rates of settlement may be up to six times faster than predicted. The values of total predicted settlement are not expected to be amended by a study of the piezometer results.

Mr G. Wilson, Director of Engineering, Port of London Authority

Mr Little has pointed out that the monoliths at Tilbury were cheap for monoliths but the question has been asked if monoliths are ever cheap. In the case of Tilbury the figures which have been given show that the ground was very poor. At the start of the work the foundations were 70 ft deep and monoliths provided the best construction, but at the north end where the foundation rises to 50 ft deep piles were possible. I might mention that on the west side of the dock on the roll-on, roll-off berth a slope of 1:5, 1000 ft long and 25 ft wide, proved to be unstable in the dry and the slope had to be flattened to 1:7.

271. Because of these conditions, coupled with the need for early completion, a simple form of construction was essential and at the time when a re-design might have been considered the Director-General asked if the job could be done in half the time. A premium was paid to the contractor to increase the quantity of plant on the site and the work was completed almost a year early, so that even if the main wall was not the cheapest construction it was completed in time and enabled customers to start work on the sites which they had rented.

272. The adequacy of the Entrance Lock at Tilbury is of first importance. At the King George V Lock Entrance there are 6000 lockings a year, which is about 20 per day. At Tilbury a number of berths have been provided and allowing for container ships to stay for two days and roll-on, roll-off ships to come and go in the same day there is a small margin. This approximate comparison, however, is not considered fully satisfactory in this important matter and the problem has been extensively analysed and there are further adjustments to be made in the lead-in jetties and the scheduling of ships. The figures are not yet complete but we have every confidence that the Entrance will be able to deal with the ships for which provision has been made. It is true that in the earlier layouts an additional lock entrance was shown, but apart from other considerations the cost of a second entrance would have made the economics of the whole scheme much more difficult. We are, however, aware of the fact that the full development and assessment of the economics of container berths have not yet been foreseen and if by any chance another entrance becomes necessary it is not impossible to construct one.

Written contributions:

Mr H. Addison

In commenting on the new impounding station, it seems to exploit to the full the advantages of the vertical-shaft axial-flow pump. Its variable-pitch propeller, in conjunction with the pump outlet siphon, enables the system to dispense with any form of main control valve—valves which can be bulky and costly. The automatic regulation of the blade pitch to maintain constant motor output, while not ensuring maximum pump efficiency under all conditions, probably gives a performance not much inferior to what can be attained with expensive variable-speed motors.

274. The absence of valves in the conduits, too, reduces the risk of undesirable surge effects when the pumps are started and stopped. In the worst conditions, when

a failure of the electricity supply might suddenly stop all the pumps, the negative and positive surges in the outlet conduits would doubtless be kept within permissible limits by the forebay into which the pump siphons discharged.

275. Because, as explained in § 130 (a), absolute reliability of the station is essential, it would be interesting to know whether at this stage a firm programme of inspection and maintenance has yet been drawn up. For example, are any precautions foreseen to prevent damage to the pumps caused by floating material? It is a question that inevitably occurs to anyone who has noticed the increasing quantities of drift-wood in the Thames. Is the protection afforded by the steel fendering around the intake heads, § 152, and by the flow straighteners in the main sump, considered to be adequate? Remembering that the passages in the flow straighteners are only six inches wide, it might be thought that any solids negotiating these spaces could likewise safely pass between the blades of the propellers of the pumps. Perhaps the chance of an unlucky bit of material being trapped just as the rotor blades are closing is too remote to be worth troubling about. As for gaining access to the sump after the stop logs have been lowered into position and the sump drained, no doubt some kind of access shaft will be provided, not shown on the small-scale cross-section of Fig. 27.

276. In regard to the periodical overhaul of the pumps, it would be advantageous if each unit in turn could be withdrawn completely from the sump, but, to judge from Fig. 27, the head-room beneath the pumphouse crane would be inadequate for this purpose.

277. Still keeping in mind the views of the pumphouse staff who will have in their charge a range of equipment of unusual complexity and unusual importance, and who, according to § 170, may be stationed two miles away from the pumps, they will naturally be interested in the most economical use of the water that their pumps have lifted into the docks system. How much of the water is likely to pass into the large 1928 western entrance lock, and how much into the original locks connecting the Tidal Basin with the main dock? In the site plan, Fig. 1, it is not quite clear whether one or more than one of these original locks is operational; and possibly arrangements here may be affected by the projected developments in the Tidal Basin mentioned in § 129.

278. But, from a wider viewpoint, it could be foreseen that when the developments at Tilbury as a whole are completed, the volume of traffic into and out of the docks system will be so great that all available locks will be used to their full capacity.

Mr R. W. Bishop, W. V. Zinn and Associates

I found the Paper most interesting but I was rather surprised that it had been decided to use such massive concrete monoliths in order to provide about 42 ft of water alongside. What other forms of construction had been considered?

280. The use of bentonite for sinking of monoliths had presumably been adopted as a result of the contractor's experience on the foundations for the Forth Road Bridge.⁷ It had considerably reduced the amount of kentledge to be handled but it had also reduced the passive pressure. Had this meant redesign to compensate for the loss of friction?

281. According to § 40, the monolith shuttering on Stage I was stripped after 18 hours and, on Stage II, after 12 hours. This was very quick; in fact, in cold weather the concrete could hardly have had time to set. Was the striking time of 12 hours specified or was this a matter for the contractor provided he accepted responsibility for the consequences?

282. In § 50, it was stated that the diver clearing out the monoliths after founding had to be careful not to disturb stability by undermining the outer edges. In the murky depths he must have had a tricky job. Any form of air-lifting—if this were possible—would presumably have tended to leach the fines out of the gravel.

283. Vessels approaching a solid quay usually did so with extra care because they

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could cause more damage to themselves than to the wall. Even so, it was surprising that a hardwood rubbing strip had not been provided. Was there any reason for this omission?

284. According to Appendix 2, the cost per sq. ft of monolith quays, exclusive of paving and services, was £10 17s 4d. Had there been any reduction in cost on Stage II observing that the length of quay was over three times that in Stage I, and furthermore, all the constructional difficulties must have already been ironed out?

Mr E. N. Davies

I should like to compliment the Authors on their attempts to measure earth pressures and would plead for more attempts of this type in particular both inside and outside cofferdams and similar sheet piled structures supported, if necessary, by CIRIA or the Building Research Station.

286. Is there any possibility that the earth pressures measured by the Authors were affected by the presence of the bentonite film?

287. An attempt was made to measure strains on the struts of the cooling water outlet cofferdam at Tilbury 'B' Power Station using a Wittemore Strain Gauge. Unfortunately the results were not very reliable although they did suggest very high loads in the top frame with much smaller loads than anticipated in the two lower frames. Considerable maldistribution was found between the two members of the battened steel struts. It was also considered that the gauge used was not suitable for the wet and extremely dirty conditions which existed.

288. I have had difficulties in the design of large, deep cofferdams in the Tilbury area, fortunately without failure but with some anxiety and appreciate the contractor's difficulties with cofferdam No. 1

289. What method of excavation was adopted for sinking the monoliths into the chalk, if this was by underwater grabbing? Was there any difficulty in digging the chalk?

290. Was the hearting concrete placed at night to suit the contractor's programme or was there some other technical reason for this? What was the workability of the underwater concrete?

Mr J. E. G. Palmer

This well-written civil engineering account of the first step of one of the two major port developments now in progress in this country contains much useful information and some excellent drawings.

292. The modern trend is for civil engineers to deal also with the economic aspects of port development proposals, and many port engineers would therefore be interested to know what annual tonnage throughputs have been forecast for the different types of berth now being brought into use at Tilbury; also what interest rate figure was adopted in order to produce the right answer from the DCF table.

293. Assuming that it was some learned economist or distinguished manager-type who pressed for a steel sheet-piled design for the Cross Berth, can the Authors say how many years 'life' has been allowed for this structure? How will it be rebuilt, when the time comes, without interfering with the operating of the port?

294. On § 3, the proposals for this mile-long West dock at Tilbury were in fact first formulated by the Chief Engineer of the PLA in his Development Report dated 31 December, 1910.

Mr J. H. Dodwell

I should be glad to know the spacing between the frames shown in Figs 4 and 7.

Mr Smeardon, Mr Newton and Mr Page

Monoliths versus sheet piled quay walls—design and cost

296. Messrs McGarey, Little and Bright queried the choice of monoliths in

preference to conventional sheet pile construction, although other possible variations, such as in situ lcos construction, were not mentioned. A review of recent major port developments will show that sheet piling is not the universal answer, although it is used extensively. The Port of London Authority has frequently used sheet piled relieving platform construction for projects in other dock groups and a review of the various merits might be appropriate.

297. Details of the head wall construction referred to in the Paper are illustrated in Fig. 42; the cost of this is £415 per ft run complete without reduction for the use of surplus materials. Although a decision to use this type of construction has been taken the details were not sufficiently advanced to enable them to be included in the Paper. The cost of a monolith quay wall in the same situation would have been marginally under £500 per ft. Peine sheet piling, which is now becoming available in this country, is probably more economic than the compound pile section used for the headwall and may marginally reduce prices for sheet pile walls.

Fig. 42. Typical section through No. 47 berth quay wall

298. A sheet pile design was adopted for the head wall as the gravel stratum was at its highest in that area and sheet piling was considered appropriate and cheaper. Elsewhere in the dock extension the gravel was generally 10–15 ft deeper, and in an extreme case—on berth 36—the ballast stratum dipped and died out completely.

299. Generally speaking, some lowering of dock water level is allowed for in the quay wall design, although with only a marginal safety factor.

300. The soil pressure exerted by the alluvium is perhaps greater than some contributors appreciate; it is considerably higher than the dock water pressure, and even when this is at maximum impounded level considerable lateral pressures have to be resisted.

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301. The monolith quay provides a 30 ft width of massive concrete capable of carrying any required load, and when a quay is built in advance of berth development this can be a great asset. With a piled quay strengthening after construction is difficult, expensive, and not always practicable. Generally, the maximum loads the quay will have to bear need to be anticipated at the time of construction. The head wall berth is designed for container operation but, due to its restricted length, will not become a major berth and substantial increases in operating loads are improbable.

302. Negative skin friction due to long-term settlement makes piling significantly more expensive in the ground condition at Tilbury whereas this factor has a stabilizing effect on monoliths.

303. It has been said that the Tilbury monoliths are cheap and it should be appreciated that they are basically mass concrete structures with only 5 tons of steel per monolith. If construction is well organized and plant-intensive the work is basically simpler than that for a sheet pile quay with a reinforced concrete relieving platform. The basic materials—cement and aggregate—are cheap, little skilled labour is involved, and it would not be unreasonable to expect monolith construction to become competitive with sheet piling on large contracts. The site organization entailed makes monoliths unsuitable for small jobs.

304. The Rotterdam quay costed at £315 per ft run referred to by Mr Bright is a comparatively light structure apparently intended for general cargo work only, using normal quay cranes. The Tilbury headwall quay is designed to carry container cranes and container handling plant with 30 ton containers stacked two high. The maximum pile load is 108 tons against 75 tons for the Rotterdam quay. In quoting Continental examples it may be pertinent to mention that the quay walls of the vast new Canal Docks of Antwerp are gravity mass concrete walls.

305. Mr Bishop refers to the use of bentonite for monolith sinking. Stability of monoliths against sliding is provided mainly by friction on the base; a small and probably temporary loss of passive pressure may have occurred in the gravel due to the presence of bentonite, but this would not be significant.

306. Where the sheet piling is normally submerged, as in the case of the head wall berth, its useful life expectation is at least 50 years, but it is clearly not as permanent as monoliths. Where sheet piling is exposed to wind and water a concrete quay must be considered marginally preferable on life expectation.

Monolith construction

307. In reply to Mr McGarey, § 218, to date there has been no measurable settlement on the monolith quays after construction. In § 242 Mr Stephens refers to loads of 250 tons and 500 tons which were in fact used to consolidate the monoliths into the ballast and chalk respectively. Details of residual sinking under this loading were average $\frac{1}{8}$ in. and maximum $6\frac{1}{2}$ in.

308. The question of drifting of monoliths raised by Mr Beckett is answered by Mr O'Donnell in § 230.

309. The additional cost of accelerating the rate of construction for Stage II was approximately 7% of the original cost.

310. In reply to Mr Bishop, § 284, there was a reduction of 2½% on Stage I rates for Stages II and III work, and the contractor was responsible for the increased costs of labour and materials.

311. Mr Haws refers to the use of steel shutters. These were kept free of ice but no other measures were taken during concreting at low ambient temperatures. In reply to Mr Bishop the Specification required monolith shuttering to be left in position for 24 hours, but this was reduced when ambient temperatures permitted.

312. Mr Bishop, § 283, refers to the lack of fendering on the quays. There are over five miles of quays at Tilbury Docks, and both initial expenditure and maintenance costs could be serious due to damage by ships and particularly by barges.

Ships coming alongside are responsible for providing their own fendering. This is standard practice in all the impounded docks of the Port of London Authority.

313. In reply to Mr Davies, §§ 289 and 290, the same grabs were used for excavating in chalk and ballast; these were Priestman double rope, 8 tye, $1\frac{1}{4}$ cu. yd capacity grabs weighing $2\frac{1}{4}$ tons. No difficulty was experienced but grabbing was naturally somewhat slower in chalk than in ballast.

314. Hearting concrete was placed at night to suit the contractor's programme for the use of batch mixers and derricks, and few men were required for this work. The sealing concrete placed underwater had an aggregate/cement ratio (by weight) of 5:2 and a 6 in. slump.

Roll-on, roll-off berths

315. In reply to Mr Williams, § 210, the bow loader at No. 35 and No. 37 berths shown in Fig. 22 was the *Travatal*, 298 ft $\times$ 43 ft beam, which was on charter to the Atlantic Steam Navigation Company. A boom was used between the ship and the side fenders of the jetty to line up the ship's ramp with the jetty ramp. No other adjustments were required to accommodate this ship.

Fig. 43. North ramp at No. 35 and No. 37 berths

316. Mr McGarey, § 224, refers to the omission of stern fendering on the jetty. The stern loaders which normally use these berths have heavy steel belting to both sides and stern. The ships berth several feet from the jetty as shown in Fig. 19, and there have been no instances of damage to either the ship or the face of the concrete ramps. The north ramp at No. 35 and No. 37 berths is shown in Fig. 43. Only ships' mooring ropes are used to hold the ships in position; it was, however, necessary to provide one additional bollard at each berth on the inside of the jetties near the derricks to secure the ships in high winds.

317. Messrs Bright and McGarey have mentioned the design loading for the jetties at No. 35 and No. 37 berths. At the design stage the maximum known trailer capacity of a major road haulage contractor was 140 tons on 4 axles, i.e. 35 units. The reinforced concrete deck was designed for HA loading plus 25%, which is approximately equal to 45 units of HB loading on spans up to and including 20 ft (see BS 153 Part 3A, Appendix A, § 6). The spacing of the jetty piles is 12 ft 9 in. The recommendations of the 1966 PIANC report have been accepted for future works.

318. The abnormal loads referred to by Mr Bright, § 250, are handled at No. 4 Crosswall Berth, in the West Branch Dock (Fig. 1), on or off the Central Electricity Generating Board's purpose-made ships *Aberthaw Fisher* and *Kingsnorth Fisher*. These ships of 2355 GRT are stern loading and moor hard against the crosswall. A

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Fig. 44. Ramp at No. 26 berth

Fig. 45. Aerial view of No. 26 berth

sectional steel ramp erected on the mass concrete quay wall connects with the hydraulically-operated deck ramps on the ships.

319. In contrast to No. 35 and No. 37 berths the England-Sweden Line operates from No. 26 berth in the south-east corner of the Main Dock, Fig. 1, where there is a shore-mounted hydraulically-operated ramp used for stern loading ships of up to 8000 GRT. Heavy stern fendering in rubber and timber as well as timber side fendering is provided on the quay walls. The ship is located exactly against the ramp and is then held by a single large diameter pin through the ramp into the sill of the ship's stern (Figs 44 and 45).

Dredging

320. The natural phenomenon to which Mr Beckett referred in connexion with dumping in the Black Deep has not changed but is now probably better understood. The Authority's maintenance dredgings have been pumped to on-shore sites for the last seven years. Similar facilities were not available for the disposal of more solid 'capital' dredgings which by their nature are not so readily transportable by tidal currents, and dumping of these dredgings in the Black Deep has continued until recently. A new dumping ground further off shore has now been negotiated at Sunk Head.

321. Temporary dumping of the reclamation gravel on the river bed, referred to by Mr Beckett, § 201, caused only minor silting around the landing stage and some additional maintenance dredging was required in this area. It was not significant in relation to the volume of the work as a whole.

Fig. 46. Shear strength of alluvium

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Soil properties

322. Numerous bore holes were sunk in the Tilbury Marsh before starting work and for some time afterwards. Properties of the strata are well understood and Fig. 46 is a typical set of results for both laboratory and in situ tests (prepared by Le Grand Adesco Ltd) for the shear strength of the alluvium. The dotted line indicates the shear strength chosen for design purposes. Subsequent sets of results were very similar and the values selected can be taken as generally indicative of the whole of the alluvium within the dock extension area.

323. There were large areas of sixty year old ash tips on the site, the highest being about 30 ft. These had sunk between 5 and 10 ft into the marsh and the alluvium in these areas could be expected to have correspondingly higher strength. The top 3–5 ft of the undisturbed marsh was of a considerably stronger brown oxydized clay occurring at about its plastic limit. It is considered that these two phenomena account for the somewhat higher strengths mentioned by some contributors.

324. The poor nature of the alluvium does not appear to be readily appreciated. Mr Wilson mentioned the two slips that had occurred; these were each about 200–300 ft long and dropped from a slope of 1 in 5 to 1 in 10. Another problem was mentioned in § 158. It is worth pointing out that the on-shore cofferdam of the impounding station was far more troublesome than those in the river. The shear strength of the alluvium is very small when compared with the overburden pressures and generates greater fluid pressure than that exerted by an equivalent head of water.

Pressure cells

325. The interest shown in these results is very gratifying and if other authorities were to carry out similar tests on other projects some useful data would emerge. The cells are rather expensive but it would be more appropriate to seek greater reliability rather than cheapness. They should be considered complementary to, rather than a substitute for, normal soil survey work.

326. Mr Haws made various suggestions regarding the behaviour of cell 46. Numerous bulk density tests were made on the alluvium, the maximum variation being 89–110 lb/cu. ft and the normal variation 93–97 lb/cu. ft. As the cells were sited within 20 ft of boreholes the soil properties mentioned were those occurring locally and differences in density might be expected to affect results by not more than 200 lb/sq. ft. The alluvium is highly sensitive and soil disturbance may have had an effect on cell readings; this is more likely to have been caused by driving of sealing piles rather than by monolith sinking.

327. The results given were based on cell readings taken continuously over a period of six months.

328. It is quite probable that pockets of bentonite or water are trapped for some time against the back of the monoliths until displaced by the greater pressure exerted by the alluvium. This would not affect the reading of the pressure cells as the liquids would be under pressure and would transmit to the pressure cells.

Shed costs

329. Major variations in shed costs can generally be attributed to different planning and user requirements. For a given set of conditions competent designers will produce comparable answers, comparatively minor variations in cost being generally attributable to design refinements, local labour costs and the competitive state of the market.

330. Mr Walpole's shed at No. 42 berth is an open-sided timber storage shed, and apart from the absence of side cladding and doors there are no loading banks, lock-ups, offices or ventilators, and electric lighting is not included in the details of costs. The costs can hardly be compared with the general cargo sheds referred to in the Paper. The Authority are currently providing a shed of somewhat similar design to

Fig. 47. Entrance lock and lower lead-in jetty

that described by Mr Walpole and the cost for this including electric lighting is £1 per sq. ft of floor area.

General

331. In § 272 Mr Wilson refers to the capacity of the Entrance Lock in reply to Mr Beckett. Fig. 47 shows the lock and the new lower lead-in jetty which incorporates a spring fender beam. This jetty will enable ships up to 500 ft long to enter the dock on ebb tides and thus increase the overall capacity of the entrance.

332. Mr McGarey refers to the use of an asphalt wearing surface on a lean mix base for shed floors. This method has been used with satisfactory results for renewing floors in old transit sheds. The floors which had sunk and cracked were broken down using lorry mounted pneumatic hammers. The broken concrete was consolidated by rolling before placing lean mix concrete, or in some cases asphaltic concrete, to raise the levels.

333. A three inch thickness of concrete, aggregate/cement ratio 8:0 and water cement/ratio 0:70, was placed on the lean mix concrete and screeded to provide a suitable surface for the asphalt.

334. Hard industrial asphalt flooring complying with BS 1076 and CP 204, 1½ in. thick, was laid by hand.

335. The rate of settlement of ½ in./year given in § 108 refers to the old parts of Tilbury Docks constructed over 80 years ago. Referring to Mr Walpole's experience at No. 42 berth, § 255, it is agreed that settlement in the reclaimed area will be much greater as shown in Fig. 25 of the Paper and Figs 40 and 41 prepared by Mr Haws, see §§ 268 and 269.

336. The piles referred to by Mr Haws are largely in water and negative skin friction was not serious in this instance. Elsewhere it is of considerable importance, its effect depending upon the thickness of the various strata. On a 22 in. dia. pile a force of up to 50 tons can be caused by 'draw-down' due to settlement in the alluvium.

337. In reply to Mr Dodwell, the spacing between frame centres shown in Figs 4 and 7 was 22 ft 10½ in.

338. The estimated annual throughputs for the various berths, referred to by Mr John Palmer, are as follows:

Timber berths, including Kraft paper, newsprint, pulp, plywood, and packaged timber	—350 000 tons
Container berths	—500 000 tons

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339. The capacity of a 12 acre site is 1800 containers each 20 ft $\times$ 8 ft $\times$ 8 ft stacked two high.

340. Referring to § 83, No. 46 berth is now being developed as a packaged timber berth. At No. 39 berth there is an additional five acre site on the west side of the perimeter road which will be used for parking trailers, offices and workshops.

341. Regarding the interest rate this is generally based on the Nationalised Industries Review of Economic and Financial Objectives (Command No. 3437, November 1967). The Government has decided that 8% is a reasonable figure in present circumstances and this rate is to apply to expenditure and receipts related to investment projects before allowing for tax.

Fig. 48. Aerial view of dock extensions, 26 February, 1968

Impounding station

342. It was impossible to indicate the full extent of the hydrological investigation within the limits of the Paper. About two thousand silt samples were taken in all and many characteristics of the kind indicated in Fig. 28 were plotted. The river reach investigated proved reasonably stable and there is no doubt a definite pattern for silt load and velocity was established. A preliminary silt survey carried out by the Hydraulic Research Laboratory on 24 March, 1959 for positions 1, 2 and 3 was compared with PLA results for these positions with a tide of similar amplitude on 19 September, 1959. The similarity of the two sets of results was remarkable and this was indicative of the consistency of the results in general.

343. It is important to remember that the investigation was concerned only with the upper two-thirds of river depth and that what was needed was an engineering assessment of the best position for abstracting water rather than a quantitative scientific analysis of silt movement in general. Subject to these qualifications Mr McGarey's remarks regarding the reliability of intermittent sampling were not borne out by the survey.

344. Two recording silt meters are to be installed in the new impounding station for occasional use to check the solids content of the impounded water. The use of

these meters for river sampling was also considered, but they were not thought to be sufficiently reliable and were inappropriate for the investigations required.

345. The soil mechanics definition of clay as particles below 0.002 mm in size is not helpful when this accounts for between 30% and 50% of the material under consideration and the further classification for fine silt given in the Paper is not unusual.

346. Apart from the anti-syphon valves no other valves were required in the culverts. It is the Authority's practice to design their impounding stations so that there is no direct connexion between a dock and the river to remove the danger of loss of water due to valve failure.

347. Surge loads will be transmitted to the bellmouth floor if a power failure occurs while the station is in operation. This was estimated as equivalent to 30 ft head of water which the floor has been designed to withstand.

348. A 12 in. $\times$ 4 in. aperture grid is provided on the river intakes to prevent flotsam entering the pumps. This aperture size has proved adequate on the Authority's other impounding stations but if needed, smaller size plastic grids could be fastened over the steel. This is unlikely to be required. An access shaft to the sump has been provided adjoining the stop log guides.

349. The pumps are operated by the vertical in-line electric motors which have first to be removed before the pump can be withdrawn. Once this is done there is sufficient head-room for the latter operation.

350. Fixed-blade impeller pumps have been used by the Authority in other impounding stations and were originally intended for the Tilbury Station. The change to variable pitch impellers is based solely on operating economics; either type of pump would have been expected to give satisfactory service.

351. The maximum water demand was calculated on the basis of two entrance locks in operation, although only one is now in use. A dam has been constructed at the outer end of the Basin Lock and leakage has been reduced to 0.2 in. per hour.

352. Mr Davies's anxiety regarding cofferdam work is appreciated. The Authority's staff measured the deflexions of the sheet piling in cofferdam No. 1, which was of the type with the clutch on the neutral axis, and the magnitude of these was such that clutch slipping must have occurred. The sheet piling was severely twisted and distorted by earth pressure to such an extent that the contractor eventually decided not to withdraw it. Arching of the soil between the struts was improbable.

353. The pump house monolith was sunk through 8 ft of chalk by underwater grabbing and, although slow, this did not prove difficult. Borehole chalk samples were recovered from the first 30 ft of chalk and visual inspection revealed no noticeable change in quality. Had the original deep-monolith design been retained it is possible that sinking through 40 ft of chalk by underwater grabbing in free air would have proved practicable.

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