Papers Nos 6412 and 6408

Underground station for Western District Post Office, London†

Design and control

by

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and

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Discussion

Mr J. H. Brass (Administrative head of Plant Department, John Mowlem & Co. Ltd) said that almost everything on the job had been mechanically handled. Plant to the value of more than £100,000 had been concentrated in a small area, and much of it had been specially designed or adapted to meet the site conditions.

140. In § 130 it was stated that "The storing of long 2-in. rods after bending would have occupied more space than was available on this site, and it was found convenient to bend and store all the steel at a central depot." That was certainly true. At one time there had been in the depot 700 tons of steel, requiring a space of 200 ft by 40 ft, and that did not include the area for the batteries for the stressing cables. The batteries had been made up from a number of 1½-in.-dia. steel tubes, accurately shaped, each more than 20 ft long and bent to 6-ft rad., twelve tubes being welded in the form of a three-dimensional skew. Special jacks had been used. The bending and placing of the tubes had been carried out to exact dimensions; but the batteries, because of their very awkward shapes had been difficult to handle, and transport from the depot to the site had been no easy matter. Some had been sent in a specially prepared lorry in a vertical position, but some in a horizontal position had had to travel at night with a police escort.

141. The depot facilities made available for the contract had relieved the site agent of many problems and had materially assisted in the execution of the work.

142. In February 1958, a tube through which a prestressing cable was to be passed had been blocked by a standard 6-ft ranging rod which had been dropped into the tube point downward. During the first attempts to clear the tube, 1 ft of rod had been extracted and the rest had been pulverized and compressed into a length of about 2 ft. The obstruction had been attacked by rods with no result, and attempts to burn it out by pouring paraffin down the tube and inserting red-hot rods had been unsuccessful because owing to lack of air the wood would not light. Eventually propane and oxygen from the normal welding apparatus and compressed air in another pipe had been used; the obstruction had been reduced to ash, which had then been extracted.

† Proc. Instn civ. Engrs, vol. 15, p. 81 (Feb. 1960).

143. In § 93 it was stated that the derricks had a 7-ton capacity and a working radius of 100 ft. In fact they had been standard derricks with 120-ft jibs, giving a load of 7 tons at 90 ft radius.

Mr F. A. Hough (Chief Regional Engineer, London Postal Region) said that Table 6 gave what purported to be the very reassuring conclusion that the design figure of 15% loss of prestress in the steel was borne out by the figures of creep and shrinkage, yet the calculations had been based on using only a percentage of the 12-month value for shrinkage and creep. Table 5 gave some test results but stopped short at 350 days, and the results did not appear to be becoming asymptotic at that time. In § 43 it was stated that the investigation was being continued. If to the above there was added the element of inaccuracy with regard to the coefficient of friction, he doubted whether in 2 years' time the stresses in the steel stressing cables would be known to within an accuracy of 2:1.

145. He then referred to one of the items by which the designers hoped to have confirmation, the strain gauges. Those gauges had cost about £3,000, including the cost of buying and having them installed and cabled. That was a not inconsiderable proportion of the cost of the end walls that the strain gauges had been designed to investigate. Also, 15% of the gauges had proved faulty early in the proceedings and more were faulty later.

146. Mr Hough then raised three questions relating to the gauges. The first concerned the design of the gauge, which had in this application shown too high a proportion of failures. He had been interested in the design at an early stage, since in essentials it incorporated techniques of which there had been some experience in the Post Office. He had not at that time examined the interior of the gauges, believing that the D.S.I.R. must know all the answers, but he had made suggestions with regard to the connecting cables. On seeing one of the gauges opened up later he had observed certain deficiencies which might well have been foreseen. Unfortunately, it was not possible to take the gauges out of the concrete walls to find out why they had failed.

147. Mr Hough's second criticism concerned the exposition given by the Authors of the results obtained from the readings. A highly unsatisfactory correction factor had to be applied from the curves of the mean of the two pairs of embedded control gauges. The mean meant nothing statistically and gave no indication of the spread between the two readings. Also, the shrinkage and creep changed by a factor of more than 2 over a year, so that the computation must take account of the time at which the readings were taken. In addition, there were the further assumptions made in § 66 to arrive at a figure of 2×10^6 lb/sq. in. for E , the Young's modulus, which was only $\frac{1}{4}$ of the long-term static value and was very significant, therefore, in its effect on the stress calculated from the strain. Therefore, when the Authors said that the strain gauges were difficult to interpret and that they served as a qualitative check rather than a quantitative one they had neither overstated the value of the gauges nor understated the difficulties.

148. His third criticism was against himself. To go round more than 100 gauges, connecting to loose tails, would have been laborious so he had conceived the plan to cable them to a central test-jack frame. Groups of leads from the gauges were to be brought to standard waterproof G.P.O. connecting boxes set in appropriate recesses in the wall, and thence connected by standard multi-core cable to the jack frame. He had failed to appreciate that there was an alternative type of connecting box which was used. It protruded from the recess, and both it and the cables had been repeatedly damaged by the building work. The mistakes in the numbering system made by the jointers had added to the troubles. It was not surprising, therefore, that the Authors had had difficulty in obtaining results. He thought that a central testing point was useful, but the connecting system must be designed to be adequately protected, for example with embedded conduits.

Mr E. C. Beck (Managing Director, John Mowlem & Co. Ltd) said that in § 1 mention was made of the original Post Office contract, which had been started in November 1914, when two contracts had been signed on behalf of Mr C. E. B. Hobhouse, then Postmaster-General, and Mr George Burt, father of the present Sir George Burt, for the construction of the shafts and connecting tunnels at a number of points on the route and for the construction of 4,265 yd of 9-ft-dia. cast-iron-lined running tunnel. The engineer had been Mr Dalrymple Hay, then a consulting engineer for the Post Office, who had later joined Sir William Halcrow, so that in a sense it could be said that the same consulting engineers and contractors who had been responsible for the original work for the Post Office Railway had carried out the contract at Rathbone Place.

150. In the bill in that earlier contract the day-work rate for a labourer, which had included supervision, establishment charges, and profit for the contractor, had been 8d/hour. At Rathbone Place it had been 5s. 7d/hour, an increase of about 1½d/hour/year. The cast-iron lining for the original contract had cost £7/ton, the corresponding figure for Rathbone Place being about £44/ton.

151. Apart from one or two small accidents the Rathbone Place contract had gone fairly well. The profit had been within 1% of the tender profit.

Mr A. Goldstein (Partner, Messrs R. Travers Morgan & Partners, Consulting Engineers) observed that from time to time engineers did their best, within some form of budget, to take strain-gauge readings on sites to prove theoretical predictions. That, of course, was an extremely difficult thing to do. It would be recalled that some years earlier there had been held what was commonly called the "Correlation Conference", and those who had tried to establish that site strain-gauge readings could be of some value had been met with criticism not unlike Mr Hough's suggestion that the conclusions from the present Authors' readings might be misleading or difficult to draw.

153. It could be argued against strain readings that this was simply another way, and a very laborious and expensive way, to ascertain the Young's modulus of concrete, because obviously any attempt at interpreting stress must make some assessment of Young's modulus. In that connexion the Authors' work was valuable, but he would raise two questions. First, the results of creep test on specimens were high—certainly higher than the values commonly used in practice. Abroad, a factor of 2 or 2½ was often used to obtain ultimate deflexions from immediate ones having been confirmed by practice. Secondly—and this might explain the foregoing—there was considerable difference between creep, shrinkage, and possibly even static modulus, between small specimens and large prototypes. This difference depended not only on size but on shape factors. The values for the prototype were usually less than those of the specimens. Would the Authors comment on this?

154. Mr Goldstein then referred to the question of stress concentration mentioned in § 28. The work on the model had been done, as he understood it, to evaluate or obtain some idea of what the stress concentration factors were, and the results had indicated a maximum concentration of 2.2 times the average applied stress across the net section at the hole. The stress concentration of an ideal case—i.e. a hole in an infinite elastic medium—could of course be readily established analytically and estimates for the case in point might have been made without much error. But what happened next? What had the Authors done with that figure of twice the average stress? Had it been used as a design stress? If not, why not? Probably because of the following. Mr Goldstein agreed that a high concentration would apply in a perfectly elastic medium, and that argument had been examined in tremendous detail in the Comet inquiry. The question was whether a hole in the concrete was analogous to a hole in a perfectly elastic medium. Even in the case of a sheet of metal, say steel, under test, there was argument that suggested that the relaxation round the edge of the hole (the plastic flow in the case of concrete) would reduce that stress. Had the Authors used that kind of argument to correlate the stress concentration for an elastic medium in terms of their structure, for otherwise, presumably, the basic design stress would have to be dropped to accommodate it. There were many examples of concrete construction where similar

holes on a smaller scale might be thought to have given rise to high stress concentrations without any or only nominal steel present. In practice no problems, cracking, or failure occurred. That was probably due to the phenomenon of creep, which, although a tremendous difficulty in other connotations, was on the engineer's side in this one. Would the Authors agree and give more details?

155. In § 51 the Authors had stated that generally the extensions had been found to be greater than those calculated, and they had given their coefficient of friction as 0.46. There seemed to be some very varied experience on that. Such an evaluation, allowing for friction, might be made as accurately as possible before the cables were pulled. But sometimes they would not pull as much as was wanted, and it was necessary to resort to all sorts of measures to get the right extension; but at other times they would readily reach the extension desired at a pressure reading that corresponded more to the minimum pressure reading, stipulated with no allowance for friction, than to the maximum, that being the top pressure to which it was desired to go. The only conclusion seemed to be that in some jobs there was very little friction indeed. Mr Goldstein believed that it had been established that the variation of Young's modulus in steel wire in prestressing was roughly within $\pm 5\%$. If that was so, the extension as calculated ought to be within 5% or so. He suspected, therefore, that the allowances made for friction were in many cases considerably greater than what was in fact necessary, notwithstanding the other occasions when it did not prove easy to get the necessary extension. Would the Authors comment on this, relating their experience in terms of greater than calculated extension to the figures for friction that had been taken?

Mr D. A. Stewart (Principal, Messrs Cowdell and Stewart, Consulting Engineers) commented on the conclusion in § 135 (v) that gap-graded concrete could be used to advantage only in the absence of reinforcement. However, quite a number of contractors in Britain were using gap-graded concrete with very considerable and concentrated reinforcement, and they should not be encouraged to think that they should put up the charge for doing so. He then described the type of concentration and reinforcement where the use of gap-graded concrete had been quite satisfactory. The maximum size of the stone had been $1\frac{1}{2}$ in. and the minimum $\frac{3}{4}$ in. In one case, a wind tunnel, the concrete had been placed into walls of 6-in. total thickness of concrete with $\frac{1}{4}$ -in.-dia. steel bars at 6-in. centres on each face in both directions. The concrete had compacted quite satisfactorily, although on mixing it had resembled a mass of stone, but when the shutters had been removed both faces had been found perfectly good.

157. One of the main difficulties was that often the bars were spaced so as to pick up their stress uniformly. If the bars were concentrated in groups so that concrete could be placed in a more satisfactory manner as well as the vibrators, the result would still be the same so far as load-carrying was concerned.

Mr L. R. Creasy (Superintending Engineer, Structural Engineering Branch, Ministry of Works) said it would be appreciated from the Papers that the Post Office Railway was one link in an intricate chain of communications designed to speed the process of handling and dispatch of mail within the metropolis. The station described had been designed to act in conjunction with the building over it which had yet to be constructed, and it might be of interest to look at the general proportions of that building and what it contained.

159. Fig. 17 was a diagrammatic cross-section through the structure, the station and tunnels being shown by dotted lines. The lowest slab, shown at about 35 ft below street level, corresponded with the roof of the station which the Authors had described. There would be two storeys in the basement to bring it to street level and then about 100 ft (six storeys) in the building above. The mails from the station would be brought to the lowest basement level and mails entering from the street would be transferred by ramps and conveyors to the same collecting point. All the mail would then be transferred to a convenient upper floor. Here it would be sorted according to destination

by a system of automatically controlled conveyors which passed through several floors and finally returned the mail to the lowest floor for dispatch.

160. The building was to be constructed of reinforced concrete, largely on grounds of economy, for reasons that he had fully discussed elsewhere¹. The column grid of the main building would be on a 40-ft-sq. panel, and three different arrangements of reinforced concrete beam framework had been examined: the "waffle" type of slab in which

FIG. 17.—SECTION THROUGH WESTERN DISTRICT OFFICE BUILDING

the main beam grid was subdivided by secondary beams; a clear drop-slab; and the conventional open-span beam and slab. Maximum economy of construction in the reinforced concrete frame coincided with the thinnest slab, and for that reason the waffle type of grid had been selected.

161. Since the building would contain a complicated arrangement of conveyors, risers, and other equipment, it was necessary to be able to cut holes in the floors if at subsequent stages there were changes in the plans. The waffle type of slab was convenient in that respect so far as the floor panels were concerned. As a further measure, tube housings for additional reinforcement would be cast into the main beams at each junction with the secondaries to facilitate the possible strengthening at a later stage. This was one of a series of devices that had been developed in providing for the adaptation and adjustment of the reinforced concrete frame, including connector plates welded to the reinforcement, straps and hangers, and the grouting-in of additional reinforcement as made necessary by subsequent alteration. As a result it might be said that the adjustment of a reinforced concrete building was not more difficult than that of other types of construction.

Mr A. C. E. Sandberg (Consulting Engineer), who had assisted on the concrete aspect of the contract, referred to the questions of concrete control, creep, and the choice of mix. So far as control was concerned, in the initial stages various aggregate sources had been investigated for quality and in particular for their ability to produce a consistent output throughout the contract. Much of the work was to be based on the test results obtained in the early stages and it was essential that those results should apply throughout the contract. A very large number of mixes had been examined and

¹ L. R. Creasy, "The economics of framed structures". *Proc. Instn civ. Engrs*, vol. 12, p. 237 (March 1959).

tested both in his laboratories and by the Contractors before the final mixes had been adopted. A site laboratory had been operated by a member of his staff whose duties had covered the normal sampling and testing of aggregate on arrival and, in conjunction with his main laboratories, the sampling and testing of cement and concrete. This tester had also been responsible for issuing batching instructions for mixes, having determined the moisture contents of the various aggregates. Cement-testing and cube-crushing had not been carried out on the site, because the main laboratory had not been far away.

163. Generally the concreting had gone extremely well, in the main because all concerned had spent some time in planning ahead and making trials, mock-ups, and modifications in advance of when they had actually been required.

164. So far as the creep tests were concerned, the initial tests and part of the main programme results were given in the Paper. The work had been a good deal more complicated than had been indicated in the Paper, which only summarized results. The first series of cylinders had been made from laboratory mixes before the start of the site-concreting in order to obtain some general results for the mixes likely to be used. The figures for creep on the first series were an estimate based only on 35-day test results combined with the general information on creep. His firm had made this estimate with some trepidation, but the forecast had proved reasonably accurate. More results were becoming available and were being put into report form. They had recently extended the scope of the tests by immersing in water the three bottom cylinders shown in Fig. 5, and the three comparable cylinders for creep and three for static modulus. They proposed to take further readings over the following year to investigate the effect of moist and dry conditions on those properties of the concrete, and would be pleased to show anyone interested the results and discuss them. There was a great deal of additional work which they would like to do on the subject, but unfortunately with the completion of the contract the useful direct application of the results had ceased, and so had their sponsor.

165. On the choice of mixes, they had had no general preference so far as the use of gap-graded or continuously graded mixes was concerned, because they considered that each job should be assessed on its merits. On the whole, they believed that within the limits of the specification the contractor should be encouraged to choose or develop his own mix and even workability, provided, of course, that the concrete did ultimately set.

Mr L. C. Callaway (Senior Engineer, Frederick S. Snow and Partners, Consulting Engineers), referring to the design of the raft in the base, pointed out that it was stated in § 18 that a longitudinal ground beam had been used and that originally the end-wall and central bases had been designed with a joint between them, but had later been re-designed so that the entire base of the station was monolithic. Presumably that had been because considerable settlement had been expected. Had the figure been theoretically calculated? If so, it would be interesting to know what it was and over what period the maximum was expected to be found.

167. Mr Callaway's firm had represented adjacent property owners and had looked after their interests while the operations were being carried out. It had been evident that all the precautions taken and methods employed in the undertaking had been completely successful. A few minor cracks discovered in the early stages had been very carefully observed during the construction and no appreciable movement had taken place.

168. The Freyssinet hydraulic flat jack used on the job was certainly ingenious and it was difficult to visualize any substitute that would have given such an even distribution of loading with such a simple method of application.

169. The building with which his firm had been concerned was of considerable magnitude and, bearing in mind the enormous excavation in such close proximity and the tunnelling operations underneath, considerable credit must be given to the designers and contractors in this undertaking.

Dr A. R. Flint (Reader in structural steelwork, Imperial College of Science and Technology, London) said that the work described in the first Paper represented the synthesis of the most advanced design, analytical, and construction techniques that were available. He had been associated with part of the investigations that led up to the final design, and would comment briefly on the model investigations that had been made at Imperial College.

171. As described in the Paper, the series of models had been tested by the photo-elastic technique, and the three-dimensional models in particular were of interest since they represented an advance in their application to post-tensioned structures. At the time of the investigations there had been no published information regarding the use of post-tensioning in conjunction with the frozen stress technique, and consequently certain preliminary tests had been required to ensure the satisfactory development of the method of applying the tensioning. The property of the Araldite B resin of contracting during set produced problems of casting, and in particular of producing a bond-free duct for the post-tensioning wires. The technique described was not without its drawbacks; in particular the coating to the fine-gauge tubing was temperamental, and there was need to investigate possible other materials which would remain soft under the oven temperatures without becoming liquid.

172. The two-dimensional models were designed to study the relative stress concentrations occurring at the fillets at the base of the stem and around the tunnels under soil and post-tension loading, respectively. The test results showed that these in fact were of the same order and that no account need be taken of the stresses in design.

173. The results of the three-dimensional investigations obtained for the soil-loaded model were entirely satisfactory despite the low "fringe order" resulting from the interaction of the side walls. Some difficulty had been experienced in analysing tests on the post-tension model, however, particularly in treating the transverse bending stresses. It should be pointed out that the investigation used well-tried techniques, and represented as reliable a source of model analysis as any available in treating such complex three-dimensional problems. Such a study naturally required considerable attention to detail, and might not be satisfactorily carried out unless sufficient time was allowed to the investigator.

174. Mr Goldstein had asked how a design should be modified if stress concentrations existed. That depended upon the nature and magnitude of the stress. If the stress was compressive, it was generally satisfactory to permit some local overstressing and plastic deformation which relieved the local concentration. If high tensile stresses existed they might be translated into local forces by integrating over the area involved, and reinforcement could be introduced to carry those forces. That was the procedure adopted in the case of corners of large cut-outs in buttress walls to a large water tank recently constructed.

175. If the structure was post-tensioned, however, the danger existed of inducing high tensile stresses during tensioning which were not exactly balanced by the compressive stresses due to the loading carried. That possibility had been foreseen in the Rathbone Place wall. The onset of a crack originating at the tunnels might have had serious consequences.

176. Dr Flint was interested in the full-scale measurements that were being undertaken on the end retaining walls, and said he would like to see a fuller description of the correlation of those measurements with the results that were obtained from models. Would the Authors elaborate on that comparison and also indicate how near the design stresses approached the measured values at the present time.

Mr R. V. Watson (Engineer, Soil Mechanics Ltd) referred to some of the minor points that had arisen over the introduction of ground loading by jacking the frame. One of the problems had been the precautions necessary to prevent a sudden release of load on the frame. If during the jacking there had been a breakage in the pipeline or something of that kind, large forces would have been released very quickly, and they might have had an effect both on the frame and on the surrounding structures. For that reason

the system of using wedges and springs had been devised; as the jacks extended, the wedges were drawn in by the springs on each side so that there was an automatic follow-up. Had there been a sudden release of hydraulic pressure there would have been very little free travel before the wedges were tight. For the same reason each jack had its individual valve.

178. During the jacking operation and at intervals afterwards strain measurements had been taken on the steel frames themselves by using a demountable mechanical strain gauge, and that had shown a close correlation between the loads as determined by the hydraulic pressure on the jacks and the loads as measured directly in the frames. The demountable strain gauge seemed to be very suitable for what he would call fairly simple measurements of that sort. It had a sensitive dial gauge, and $7\frac{1}{2}$ divisions on that gauge corresponded to 1 ton/sq. in. in mild steel, so that for the stress range which had to be dealt with in temporary works a sufficiently accurate measurement was obtained. It was not expensive and nothing valuable or expensive had to be left on the job; only small holes had to be drilled in the member concerned, into which two points on the gauge fitted. As a matter of technique, the holes had at first been drilled into the mild steel of the structure but that had been found not entirely satisfactory, for two reasons; the surface of ordinary structural steel was rough, so that a neat hole was not obtained, and it was inclined to rust. Towards the end of the job stainless-steel drilled disks had been stuck on the mild-steel structure by means of Araldite adhesive, and weathering tests in the last two winters seemed to show that these disks would stick to mild steel very well for one winter but needed protection if they were to be used for longer.

179. On the question of prestressing, Mr Watson endorsed the views of the Authors on the use of precast anchorage units. It avoided the need to fix small reinforcement among the larger-diameter reinforcement of the main structure, it avoided the handling and fixing of a large number of anchorage elements, and it allowed more curing time before the highly stressed concrete around the anchorages was loaded. It also made it possible to take special precautions in compacting the concrete on the precasting table round the anchorages themselves. There had been no failure in the anchorage blocks throughout the job.

180. Finally, they had been faced with the problem of grouting the cables against a 60–70-ft head. A special grouting nozzle had been developed for holding the grout pressure while a wooden bung was driven in, so as to prevent the loss of grout.

Mr J. R. J. King (Deputy Agent, John Mowlem & Co. Ltd.) referred to the tunnel construction which had gone on throughout the period under discussion. A prominent feature of the layout of the tunnels had been the exceptional number of changes of diameter in a relatively short length of drive. Added to that, the tunnels between the junction chambers were laid out mainly to horizontal or vertical curves with transitions, and in some cases curving horizontally and vertically together. That had necessitated concentrated supervision of the setting-out, and constant checking of the alignment of tunnel rings and of the timbering at the various changes in diameter. The tunnels had been driven throughout in London Blue Clay, under property, by normal hand-mining methods.

182. The most interesting feature was the terminating step-plate junctions, Nos 1 and 4, which had had to be constructed round the old 9-ft running tunnel without interference to traffic running every few minutes through the tunnel. The old tunnel had been carried on timber cribbing, this being replaced by concrete infilling every 20 ft. On completion of the terminating chambers the Contractor had been granted several week-end possessions of the track, during which they had made preparations for the diversion of the tracks from the old alignment to the new. During one of these 36-hour possessions the old tracks had been completely removed, the track beds cast, and the new tracks set and concreted in. The first train had run through the new tunnels and station on the Monday morning at the appointed time. Immediately after diversion, the old tunnel had been dismantled where it had passed through one corner of the station excavation, so that that part of the station construction could be completed.

183. Some of the broad flanged beam piles had been driven fairly close to the old 9-ft-dia. tunnel. Those that had been driven to 12 in. above the crown had had no noticeable effect on the lining, whereas those driven to 15–18 in. to one side had caused some slight lateral deformation. As a precaution when driving piles close to the running tunnel driving had been stopped when the pile toe was within 10 ft of the lining and continued only between 8 and 10 a.m., when the Post Office carried out track maintenance. An engineer was then stationed in the tunnel with telephonic communication to the site while driving was completed. It had been possible in each case to drive the pile to its final destination without alarm.

The following contributions were received in writing.

Mr G. M. Cornfield (Senior Engineer, The British Steel Piling Company Ltd), referring to the earth pressures exerted on the temporary walls of the excavation, asked the Authors to indicate the values of the maximum waling loads, in tons per foot run horizontally, that were used for the design of each of the steel frames. How did those design figures compare with the corresponding measured values of the waling loads? Was it possible, from observation of the jack pressures or from the readings of the strain gauges attached to the steel framework, to determine the actual waling loads per frame under the following separate conditions: first, just at commencement of jacking; second, at completion of jacking when the excavation level was only just below the particular frame. Moreover, what were the corresponding measured loads in each of the frames after the excavation had reached final level?

185. With regard to the question of jacking and earth pressure, it was not clear how it was possible to determine whether the soil was really in the “at rest” condition—could not partially active or even partially passive pressures have been developed after jacking had been completed? Presumably, some initial inward soil movement had occurred while excavating down to the level of a particular frame before that frame had been inserted and jacked.

186. In § 101 it was stated that a figure of 1.55 ton/sq. ft was taken for earth pressure at rest, and 1.1 ton/sq. ft for active pressure. Were both of those figures equivalent to the width of the trapezium of soil pressure distribution given in the Code of Practice for earth-retaining structures?

187. Since this was one of the very few cases where measurements had been taken of pressures due to the soil in an excavation in London Clay, further information regarding the results would be extremely valuable.

Mr T. N. W. Akroyd (Chief Engineer, Constructional Services Ltd), referring to the comparison of gap-graded concrete with continuously graded concrete which was made in both Papers, observed that the contract was one of the first on which a modern specification for concrete had allowed the Contractor not only to design suitable concrete, but with the approval of the Consulting Engineer to use either gap-graded or continuously graded concrete as required. Would the Authors give some further information on the long-term tests on both types of concrete?

189. In § 39 it was stated that to achieve high density and minimum shrinkage the use of gap-graded aggregates had been specified. That implied that those requirements could be achieved only with gap-graded concrete. That followed, of course, from theoretical considerations, but owing to the impossibility of achieving the theoretical limit of gap-grading of the aggregate the voids to be filled were greater than the theoretical minimum and more mortar was present than was actually required, so that in practice gap-graded concrete was not free from drying shrinkage nor from moisture movement. That was borne out to some extent by Table 3, where the difference in initial drying shrinkage and moisture movements between the two concretes might be expected from the difference in the aggregate/cement ratio of 6:1 and 7:1. From a consideration of the initial drying shrinkage and moisture movement figures given in Table 3, and the coefficient of creep given in Table 5, it appeared that very little advantage was obtained

in the hardened concrete from the use of gap-graded aggregates. Would the Authors say why they persisted in its use?

190. The main disadvantages in using gap-graded concrete stemmed from practical considerations. Gap-graded concrete was sensitive to the changes in the aggregate gradings and was particularly sensitive to changes in the specific surface of the sand. It followed that a slight change in size of one of the aggregates could completely alter the characteristics of the concrete. Similarly, the relation of the aggregates depended upon their shape; the substitution for a rounded aggregate of one of the same size, but which contained crushed material, would also affect the resulting concrete. Uniformity was more critical with gap-graded aggregates than with continuously graded aggregates.

191. The quantity of water added to a gap-graded concrete was critical. If insufficient water was added then complete compaction could not be obtained. On the other hand, if too much water was added wet segregation would follow. This sensitivity to water content could be an advantage or a disadvantage, depending on the site control. If the site control was good then the sensitivity of the concrete would form an excellent means of achieving consistent concrete. On the other hand, if the site control was only fair or even poor then failures could be expected.

192. Difficulties experienced in the vibration of gap-graded concrete resulted largely from the greater proportion of large-sized aggregate used in gap-graded concrete. Such large-sized material required a greater energy input to bring it into the same state of vibration as smaller particles, although it had been urged by some exponents of gap-graded concrete that on vibrating there was an immediate collapse, so that less vibration would be required. If this was so, could the Authors of the second Paper enlarge upon their statement in § 135 that gap-graded concrete could be used to advantage only in the absence of reinforcement?

193. Referring to § 41, Mr Akroyd pointed out that a VB consistometer usually gave misleading results when it was used to measure the workability of gap-graded concrete, because of the difficulty of determining the end point in the test; this was shown by the results for workability in Table 3.

194. The results in Table 4 showed a higher standard deviation and coefficient of variation for gap-graded concrete, together with lower mean strengths compared with continuously graded concrete with corresponding water/cement ratios, thus indicating that gap-graded concrete was more difficult to control than continuously graded concrete.

Mr Roderick Cameron (Civil Engineer, Site Control Branch, Ministry of Works) observed that although expanded metal as stop-end shuttering undoubtedly gave a good structural key it was not suitable for watertight construction because it was impossible to compact the concrete fully against the expanded metal. Therefore there must be some voids, and he would expect some leaks if water was present in the ground. If a fully watertight structure was required the stop-end should be grout-tight so that the concrete could be fully vibrated.

196. Were the Authors objecting to gap-graded concrete or to the particular mix they had had to use? An extreme type of mix where the large stones were more or less touching would, no doubt, be difficult to place although it would achieve minimum shrinkage. But undoubtedly the 6:1 mix would have been perfectly satisfactory if the grading had been $\frac{3}{4}$ -in. single-sized aggregate with an appropriate quantity of sand. Gap-graded mixes of that type were widely used in reinforced concrete work. The big advantage, particularly on medium-sized jobs, was that the right grading could almost certainly be obtained, whereas with fully graded aggregate the result was often $\frac{3}{4}$ -in. concrete in places and a somewhat hungry $\frac{3}{4}$ -in. gap-graded mix in others.

Mr A. P. Cumming (Senior Engineer, John Mowlem & Co., Ltd) remarked that the Papers described the design and construction of a massive underground building as though those two problems were almost unrelated. The designer of the permanent work had spent a long time and had gone to a lot of trouble to investigate very thoroughly the stresses in the permanent work, but the design of an entirely separate

system of temporary support to the ground became the Contractor's responsibility. Had consideration been given to the use of the temporary supports as part of the permanent construction? It was quite common practice for consulting engineers to invite alternative proposals from contractors.

198. In a competitive tender, in which the complexity of design was a major consideration, it was difficult to see how the ideas of the contractor could be co-ordinated with the requirements of the consulting engineer, although such co-ordination could result in a saving to the client. At Rathbone Place the Contractor did put forward an alternative scheme for the tunnel work which had resulted in a substantial saving. Time for tendering had not permitted such an alternative scheme for the station structure.

199. About 180 tons of steel had been used for each of the four frames and a further 400 tons for the King piles and soldier piles, i.e. a total of about 1,000 tons of steel which was used for a short time and then either left in place, but without value to the permanent structure, or removed in the hope that it would be made use of again. The designer of the permanent work was not in a position to co-ordinate the design of permanent work and temporary work, since the first had to proceed almost to finality before the inquiry was issued, and the consulting engineer would not wish to restrict the freedom of the contractor. Once the contract was let, neither consulting engineer nor construction engineer wished to introduce more variations than necessary, and certainly under the present contract system they would not want to redesign the whole work.

200. At Rathbone Place the steel frames were located just above the permanent floor levels; precasting techniques, and contractors' plant, were now quite capable of supplying large concrete struts and handling them into place; could these have been designed so that they formed the beams of the permanent floor? Alternatively, steel struts might be cased in concrete and used as beams in the permanent work. Similarly, the King piles might perhaps have been driven in the centre of the future columns and used as reinforcement, any "out of plumb" being adjusted by altering the arrangement of mild-steel bar to bring the centre of steel area to the centre of the columns. Alternatively, they could be accurately placed in boreholes carried to basement level, and only driven for penetration below this level. Since the soldier piles were able to span from frame to frame supporting the pressure due to the ground at rest, they could equally span from floor to floor and could support a skin of concrete to give a clean appearance with the concrete spanning between the soldier piles. Such a combination of permanent and temporary work would have eliminated any risk of ground movement due to transfer of loads from temporary frames to permanent supports.

201. One way of obtaining co-ordination of design and methods of construction was to call the contractor in at the early stages of the design before competitive tenders were called for. That was difficult when it was required to get tenders from a number of contractors, for they might each have different ideas of design and methods of construction. Had the Consulting Engineers considered how this co-ordination could be achieved while retaining the advantage of the consultant/contractor system with competitive tendering?

Mr Edward E. White (Executive Vice-President, Spencer White & Prentis, Inc., New York), who had seen the excavation referred to in the Paper, observed that it was in many ways similar to several deep cuts in soft clay constructed by his firm in Chicago in 1955 and 1958 and it was interesting to compare the different techniques used in London and in the United States.

203. Soldier piles had been used in Chicago as in London, although not on a batter. However, it had not been necessary to use steel sheeting on the upper level, since the clay ran to within a short distance of the ground level and there was very little water perched on the clay. There had been, however, a troublesome source of water from leaking sewers and water lines that had taken a long time to correct.

204. The earth between the soldier piles had been sheeted by horizontal wood sheeting rather than concrete. In the United States the demolition of old buildings to make way for new structures provided a large quantity of excellent secondhand 3-in. and 4-in.

floor joists for horizontal sheeting at a relatively cheap cost running only about £21 per 1,000 bfm.

205. The clay in Chicago had an unconfined compressive strength of 0.3 ton/sq. ft and would not stand unsupported for a height of 3 ft until the concrete walls were placed. The clay was easily trimmed and the planks could be sawn to take care of the variation in spacing of soldier piles. Care was always taken to leave a space or louver of about 2 in. between boards to provide drainage and permit repacking behind sheeting if necessary. The 3-in. planks were used for the entire depth of the cut, for experience had shown that the ground arched from soldier pile to soldier pile and only one portion of the load was carried by the plank.

206. Precast concrete planks presented a serious difficulty because of the variation in spacing of piles and if timber was expensive the concrete walls, as described in the Paper, would give excellent support. In water-bearing ground care had to be taken to provide drainage by weep holes, otherwise water pressure would build up against the concrete, increasing the loadings on the bracing system. One of the advantages of horizontal louvered sheeting was that that could not happen.

207. Prestressing the bracing with Freyssinet jacks was ingenious and Mr White did not know of their use in the United States on excavation work. They provided a cheap and easily handled jacking system. The usual method in the United States was to use two hydraulic jacks, each with a capacity of 50 tons reacting against clips welded against the braces (Fig. 18). However, the prestressing loads were not as great as those des-

FIG. 18.—DETAIL OF PRESTRESSING ASSEMBLY

(Wedges placed at the end of the brace take the load to release the jacks)

cribed in the Paper for strain-gauge measurements showed that the prestressing loads amounting to 25% of the design load were dissipated into the soft clay within 24 hours and that only about 50% of the prestressing load remained in the braces. It had also been found that temperature had very little effect, for temperature stresses were also dissipated into the clay until the permanent concrete wall had been poured.

208. The designers showed commendable foresight in providing for extra capacity in the braces to take care of accidental damage to struts. That was often neglected and there was a great possibility that the entire system would fail if only one brace was damaged.

209. It was interesting to note in the conclusions that the bracing loads had been difficult to determine accurately. That was also true in the United States. In Chicago a careful check of the load in the braces had been made with a strain gauge to measure the actual loads in the bracing. These readings had proved to be of great value during the progress of the work and should be standard procedure on large-scale excavation work as a guide to future work. The measurements taken by Dr Ralph Peck on the Chicago subway in 1939 had been the basis for the design of the Inland Steel job in 1955 and the measurements taken on Inland Steel had been used on the Harris Trust work in

1958. An interesting comparison between the design loads and actual loads on the Harris Trust Co. excavation was reproduced in Fig. 19. The total load had been accurately determined; however, the distribution was not correct with a greater load in the top braces than estimated and a lesser load in the bottom brace.

210. Soil mechanics had been too frequently confined to small-scale laboratory tests and every opportunity should be taken to use large-scale tests to advance the science of soils engineering.

Mr Stuart Blackford and Mr E. W. Cuthbert, in reply, observed that Mr Hough regarded Table 6 as being somewhat speculative, but in their opinion the figures of loss of prestress were conservative. Even if the percentage error in the estimates of creep or shrinkage were large, the error in the assessment of loss of prestress would be only 1 or 2%. The actual loss would largely depend on the shrinkage and creep of the concrete at the back of the wall in contact with the ground, and this would always be difficult

to assess even if all the strain gauges worked perfectly. While the creep at 350 days did not appear to be asymptotic, the Authors were confident that the final strain would not be excessive. The result of checking the elongation of cables during stressing had confirmed that the allowance for friction in the design had been adequate. The long-term stress in the steel was certainly known to a considerably higher degree of accuracy than Mr Hough's 2:1, since it was only in assessing the relatively small change in stress from the known initial value that assumptions had to be made.

212. More than 15% of the gauges had failed, but they and their wiring had been subjected to very severe treatment during concreting. The application of the correction factor determined by averaging results obtained from embedded control gauges was the best that could be done in the absence of a special investigation into the behaviour of such gauges, and was to a very great extent confirmed by other gauges as noted in § 65. The correlation of observed and calculated values referred to in § 70 was further confirmation. The choice of the value of Young's modulus of 2×10^6 lb/sq. in. was as described in § 66, and in the Authors' experience this was the best available estimate, although higher values would apply to strains observed shortly after applying loads or to strains corresponding to short-term loads. The assessment of Young's modulus in such cases was difficult and uncertain. The Authors agreed that improvements could be made to the design of the gauges, particularly in the leads between them and the face of the wall. However, the problem of taking readings during construction was the greatest single difficulty.

213. Mr Goldstein, who considered that the results of the creep tests were high, had suggested that there was considerable difference between the properties of small specimens and large prototypes. The Authors had had no experience of determining the values of creep, shrinkage, and static modulus direct from a large prototype, but the factors mentioned in the reply to Mr Hough were relevant.

214. The tests on two-dimensional models had been carried out to check stress concentrations around tunnel openings and at the toe and heel fillets. The Authors agreed that where the concrete was in compression around the tunnel openings, stress concentrations might be safely diffused by creep with some controlling steel added at points of high concentration, but, as Dr Flint had mentioned, where tensile stresses were indicated additional reinforcement had to be provided. Three-dimensional models had been used partly to confirm the design assumption that the wall stem would act as a pure cantilever. It would be seen from the Paper that it did not, and the reduced stresses greatly mitigated the problem of stress concentrations. In the design they had kept to the cantilever method because they were balancing an earth moment against a pre-stressing moment, and although the model results indicated lower stresses the balance was still required. The three-dimensional-model results had been taken into account in dealing with transverse and other stresses. To take excess stresses determined from the two-dimensional model results, additional tensile steel had been provided around the tunnel openings and compression steel at the root fillet on either side of those openings.

215. The coefficient of friction of 0.46 applied only to short, small-radius reverse curves, but elsewhere the coefficient approximated to 0.35. Consequently, in the west wall where the cables were not formed on a helix the elongations varied from 9% less to 20% more than those calculated, the average being about 9% more. In the east wall where the cables contained a wire helix the elongations were 5–10% higher, owing, the Authors believed, to the collapse of the helix at the small-radius bend at the foot of the wall.

216. Mr Stewart had given an example of the successful use of gap-graded aggregate in reinforced concrete. The Authors considered that the difficulties of placing and compacting the very congested sections at Rathbone Place were greater, but agreed that the concentration of bars into groups was useful; this had been done where practicable.

217. Mr Sandberg's offer to show the results of further creep tests would be gratefully accepted. The Authors fully agreed that the specification should give the contractor scope to develop the most suitable concrete mix for the job.

218. Mr Callaway had mentioned the design of the raft. The base had originally been designed with three transverse joints, one at each end wall base and one in the central section. These joints had been eliminated in order to avoid high concentrations of pressure at the ends of the wall bases and possible incompatible movements of the base and floors above. The base was designed to act either as a beam on elastic foundations for a restricted width on each of three lines of columns, or as a rigid base. For each case the maximum deflexion had been calculated, and it was considered that here there was little practical difference between them.

219. In reply to Dr Flint the Authors regretted that at present they had insufficient information to elaborate on the comparison between the behaviour of the models and the prototype.

220. Mr King had referred to the distortion of the existing tunnel during construction. The Authors considered that this arose from distortion of the mass of the ground surrounding the tunnel during excavation for the station rather than from the effect of driving the piles.

221. In reply to Mr Akroyd, the Authors had no information to add to the 350-day results for the concrete. The figures in Table 3 indicated that for gap-graded and continuously graded concretes of comparable strength the initial drying shrinkage and moisture movement were 20–25% less with the former than with the latter, the creep being almost the same. These results confirmed the Authors' view that there were advantages in using gap-graded concrete, but the contractor had been permitted to use continuously graded mixes where placing had been difficult. Considerably more continuously graded concrete than gap-graded had been used, and for the aggregate/cement ratios involved the higher coefficient of variation of the latter was to be expected.

222. Mr Cameron had commented on the use of expanded-metal stop-ends. The Authors' experience was that although the trouble referred to did sometimes occur it could be dealt with, and usually the concrete was fully vibrated. It was always desirable to strip the expanded-metal stop-end; if a small mesh was used this could be done easily, and a good face requiring very little treatment was obtained. The gap-graded mixes used had been satisfactory, but the extreme congestion and difficult access had demanded continuously graded mixes in many places.

223. Mr Cumming had raised a very important point regarding the co-ordination of the design and construction of large works. Under the design-bid system there should be no difficulty, but where comparative tenders were called for it was more difficult to ensure that the design not only met the requirements of the client but was well adapted to the contractor's methods. It was not possible to call in any one contractor for formal discussions, and in the Authors' view it was the responsibility of engineers to have possible methods of construction in mind at the design stage.

224. In the particular case of Rathbone Place Mr Cumming's proposals appeared attractive, and the Authors were very conscious of the large weight of temporary steelwork that had been required. The levels of the two permanent floors had been fixed by the Client's requirements. Assuming two of the frames for supporting the ground were located at these levels, it would have remained necessary to provide two temporary frames above them and one, or possibly two, temporary frames below them to keep the spans of the soldier piles within safe limits. The longitudinal struts required for the frames incorporated in the permanent floors would have had to be removed to permit the end walls to act as designed and to clear the openings for conveyors, lifts, and stairs. The incorporation of the walings of these frames into the permanent work would have led to difficulties in designing wall reinforcement to give continuity with the floors.

225. One line of king piles could have been arranged to coincide with the permanent columns on the centre line of the station, but other lines required for the support of the temporary frames and permanent floor beams would have obstructed the platforms or tracks and would have been temporary.

226. During construction consideration had been given to making permanent use of the soldier piles. The piles could not have been cased with structural concrete

without excavation around them, which would have released the ground and destroyed the reason for their use. The Authors were not satisfied that unprotected steel could be relied on for a structure intended to remain in use for many years.

227. The Authors concluded that if the station had been designed from the outset on the lines suggested by Mr Cumming some saving of steel might have been effected, but without actually preparing a detailed scheme it would be impossible to be sure that overall economy would result.

228. The Authors were grateful to Mr White for the data of similar works in the United States of America, and particularly for details of the differences in the behaviour of the soft clay as compared with London Clay.

Mr V. H. Collingridge and Mr R. E. Tuckwell, in reply, remarked that some explanation was due to Mr Stewart for the bald nature of the conclusion regarding gap-graded concrete, which could certainly be placed under conditions such as those illustrated by Mr Stewart. For satisfactory placing it was essential to be able to drop the concrete almost directly into position without subsequent lateral movement. In the work at Rathbone Place much of the concrete had been in slabs several feet thick with high intensity of reinforcement in top and bottom mats of bars and in batteries of cables throughout the depth, as could be seen in Fig. 15.

230. Gap-grading had been preferred to continuous grading except in situations where lateral movement between layers of cables and rods had been necessary to get the concrete into place. A 3-ft trial "mock-up" cube with a typical arrangement of bars and cables had been made very early in the contract to check the effect of placing gap-graded concrete in these conditions. When the cube had been turned over, the difficulty of placing this concrete under the intersections of 2-in. bars at 6-in. centres in the bottom mat had been revealed.

231. Mr Sandberg had provided an opportunity for the Authors to remark on the great value of a site concrete laboratory under the control of a trained technical assistant. The size of a contract would always dictate when such a facility was feasible, but it was essential on work of magnitude where high quality and changing mixes were demanded. In the present instance the engineers had eliminated the possibility of concrete control becoming the victim of the competitive pruning-knife by specifying exactly what they wanted in the way of laboratory equipment, staff, and testing routine, and by including an item in the Bill for the provision of these services.

232. With regard to Mr Ackroyd's reference to the vibration of gap-graded concrete, the Authors' experience had confirmed that a batch of gap-graded concrete of low water content would collapse immediately on vibration; and that if this vibration did not bring the material into its final resting place, no amount of further vibration would shift the mass elsewhere, for instance under a battery of cable ducts or through a mat of bar reinforcement.

233. The Authors hoped that they had not given Mr Cameron the impression that they had raised objections to the use of gap-graded concrete except where lateral movement was required during the placing process. There were many advantages, not the least being the ease with which men could walk on concrete that was being placed.

234. Mr White's account of comparable work in the United States was most interesting, especially in the similarity of approach to the problems involved. When the work at Rathbone Place had been planned, special attention had been given to the exclusion of percolating ground-water, either natural or from service pipes, etc. Considerable embarrassment had been foreseen, especially in dealing with the overhanging clay faces on the two ends of the job. To avoid trouble from this source, the engineers had specified steel sheet-piling to "cut-off" in clay, and the Contractor had chosen in-situ curtain-wall construction against the clay principally for three reasons. First, to exclude any ground-water which might be migrating in the "backs" and fissures in the clay; secondly, to prevent such water from softening-up the clay; and thirdly, to ensure the uniform spread into the ground of load from the stressed strutting which was essential

in taking advantage of the high-load short-travel characteristics of the Freyssinet jacks.

235. In reply to Mr Cornfield's question in § 186, the design loads for the needle joists, and hence the walings, were based on a trapezoidal distribution using that given in the Code of Practice for earth-retaining structures for the "at rest" pressure; for the active pressure the distribution given by Peck² had been used. The values of 1.55 and 1.1 ton/sq. ft given in § 101 were equivalent to the widths of the trapeziums in each case.

236. The calculated loads in tons for the needle joists, which were generally at 5-ft-3-in. centres along the walings, were as shown in Table 8.

TABLE 8.—CALCULATED LOADS IN TONS FOR NEEDLE JOISTS

Frame	East wall	West wall	North and south walls
Flying waling	—	57	—
1	138†	81	Waling load/ft: 6.6 tons*
2	86	86	" " 11.5 " *
3	86	86	" " 64
4	102	102	64

* There were no needle joists along these walings.

† Combined reaction of west end of frame 1 and flying waling.

237. Before the commencement of jacking each frame the loads on the walings had been zero. At the completion of jacking each frame the loads on the walings had been those that had been deliberately jacked into the needle joists. Those applied loads were the values given in Table 8 adjusted for temperature as indicated in § 115.

238. Some strain-gauge measurements had been taken on the main frame members at intervals after jacking. These indicated variations in the stresses in the members from those measured immediately after jacking. Variations were recorded of between 40% below to 30% above the original stresses adjusted to a temperature of 70°F. Insufficient measurements had been taken to attempt a conclusive analysis of these variations, but three major factors were indicated:—

- (i) Variations in temperature of the steel frames.
- (ii) The effect of jacking the frames below.
- (iii) Behaviour of the soil mass behind the retaining walls.

239. As Mr Cornfield had suggested, it had not been possible to determine whether the soil was, in fact, in the "at rest" condition, but it was considered that the outward movements during the jacking operation followed some compression of the ground, even if passive pressure had unavoidably developed initially.

240. Taking some account of the stress variations which might be expected owing to change of temperature, the following conclusions had been reached about the behaviour of each frame after jacking:—

- Frame 1. A small reduction in stress followed by a return to approximately the original values.
- Frame 2. A small reduction in stress (about 10%).
- Frame 3. An increase in stress of about 20% before jacking frame 4. (This may have been partly due to the additional depth of ground the frame had to support before jacking frame 4.)
- Frame 4. No measurements had been taken after jacking.

² R. B. Peck, "Earth-pressure measurements in open cuts, Chicago (Ill.) subway". *Trans Amer. Soc. civ. Engrs*, vol. 108 (1943), p. 1008.

241. Strain-gauge measurements were taken only up to the commencement of jacking frame 4 (about 4 months after jacking frame 1), and therefore did not give an indication of possible long-term effects. It seemed, however, that the pressure values assumed were at least of the right order.

242. The Authors were very conscious that further detailed information would have been useful. Unfortunately, the sudden change in tempo on the contract—from a simple excavation to a complex reinforced concrete construction—made such demands on the site staff that further investigation could not be undertaken. That was not an uncommon experience in contract work.

CORRIGENDUM

Proceedings, August 1960.

p. 480. Fig. 12, referred to in § 45, line 1,
was omitted and is now printed below.

FIG. 12