DISCUSSION ON NEW STEELWORKS,

E. Edgar Williams & Sons. Painting.
J. C. Hitt & Sons., Ltd. Heating ancillary buildings.
Bolton Gates Co., Ltd. Gates and sliding doors.
Wilfred Robbins, Ltd. Rolling shutters.
Carter & Co. (London), Ltd. Tiling.
L. Mills & Co., Ltd. Tiling.
Concrete Ltd. Patent pre-cast flooring.
Fabrigerete Products Ltd. Patent pre-cast flooring.
Etchells, Congdon & Muir, Ltd. Lifts.
Lenscrete Ltd. Roof-lights.
Johnson Floor Co., Ltd. Granolithic paving.
Ragusa Asphalte Paving Co., Ltd. Asphalt.
Fleximastic Ltd. Acid-resisting asphalt.
Acalor (1948) Ltd. Acid-protection.
Prodorite, Chemical Engineers & Consulting Contractors. Acid-protection.

The Paper is accompanied by twenty-seven photographs and twelve sheets of drawings, from some of which the half-tone page-plates, the folding Plates, and the Figures in the text have been prepared; and by four Appendices, two of which have been incorporated in the text.

Discussion

The Authors introduced the Paper with the aid of a series of lantern slides and a film.

Mr H. V. Hill, referring to the expansion of the gantry girders (p. 23), said that he did not completely understand the manner in which the gantries were mounted on the columns. The Authors had stated that, if the average temperature were about 70°F., the columns should be vertical; but, according to his own rough calculations, the last column, which he presumed to be about 440 feet from the centre-line of the braced column, might move as much as 2½ inches; that was to say, a 2½-inch error might be anticipated.

The Authors had made reference (p. 24) to 11-inch cavity brickwork. What was the height of the wall and how was it supported? There was a rubber insulation between the top of the wall and the sheathing above. Had any means been found to fix or partially support the wall from steelwork, or had it been necessary to erect an independent wall with brick piers?

The difficulties of welding such massive girders could easily be appreciated, and he believed that thermit welding, which had been used successfully in shipyards for very large pieces of steelwork and for repairing large castings on ships, would have repaid some investigation.

Could the Authors give more details of the procedure used in the arc-welding process? What sequence of welding had been adopted? The last question was prompted by the statement, made in the Paper, that five welders had been working at the beginning and that their number...
had been gradually reduced to two; the weld sequence was obviously of great importance, to reduce the shrinkage. Had the shrinkage actually been measured?

Referring to the apparatus used for the detection of cracks in welds, how was it decided if a crack, when detected, was a serious one or merely a local area of porosity?

Mr Hill drew attention to the last paragraph of the section headed "Loading Test on Structural Steelwork" (pp. 18–20). His own experience with such testing confirmed that a great deal of work had to be done in preparation for it.

In connexion with the test, what relation did the vertical and horizontal loads bear to the actual maximum loads to which the structure would be subjected?

Dr L. J. Murdock said that when he had visited the site, before construction had commenced, a Bren-gun carrier had been the recommended means of transport. That gave some indication of the initial site conditions.

The Authors had given details of the settlement under the brick-store, and he thought it might be interesting to attempt a closer estimate of the settlement than that given by the rough rules in the Paper. The original consolidation-tests on the peat and on much of the subsoil at Abbey Works had been made on specimens 4 inches diameter by 3 inches thick, because, in the smaller specimens normally used, the interference from the woody material in the peat and from the stones in the subsoil had been liable to cause considerable errors in the test-results. It had been estimated that the settlement under the brick-store, in the hydraulic filling, would be between 9 and 10 inches and that considerably more than 90 per cent. of that settlement would be complete within 15 months. A settlement of 10 inches had actually occurred, but he thought that the estimate had been reasonably accurate, in view of the assumptions which had had to be made in such tests and the variability of the peat-layer.

The Authors had stated that the use of $\frac{3}{16}$-inch limestone dust instead of channel grit, in order to provide the coarser sand-particles which were absent from the dune sand, had necessitated a higher water-content, in order to produce sufficient workability. Had the increased drying-shrinkage resulting from the increased water/cement ratio produced any noticeably greater width at contraction joints, or had there been a greater tendency to shrinkage-cracking?

Dr Murdock had had an opportunity of analysing, in detail, the expanding aggregate, which came from the United States of America, and it appeared to be composed of about 90 per cent. finely granulated iron, 10 per cent. calcium chloride, and a very small quantity of detergent. Some tests had been made to determine any expansion which might have occurred during the setting period, and, although those tests had been difficult to carry out and of doubtful accuracy, his interpretation was that any...
expansion, either during the setting period or subsequently, was extremely small and that a good cement mortar was just as effective. That was, of course, an opinion which had originated in a laboratory, and he would be interested to hear what had happened under the actual conditions.

He noticed that the deviation from the average of the strengths of the concrete test-cubes, given in Table 6, was within 3 per cent. of the values obtained on a number of other sites, where similar types of mix and methods of proportioning the aggregate had been used, but where conditions of cement, aggregates, and labour had been different. He thought that that indicated the value of assessing those variations for any particular type of mixing plant in order to predict the variations and to use the knowledge in providing effective control. If an investigation were made when any cube-results varied from the deviation given, whether they were too high or too low, it was invariably found that something was wrong; usually it was the proportioning of the materials, but sometimes it was unsatisfactory cube-making, curing, quality of cement, or some other factor.

Mr J. M. Fisher observed that the question of concrete control had been given considerable prominence. The regular laboratory testing of concrete in various sections of the work had added to the increasing fund of knowledge of the subject of uniformity of concrete-quality. His personal endeavour was to produce concrete of the uniformity that was normally expected in steel; he thought that that ideal would be attained eventually. A greater degree of quality-control would lead to reduced costs and—an important point in view of the heavy consumption of first-quality concrete materials—it would permit the use of inferior materials.

Figs 27 showed the marked superiority of weigh-batching over volume-batching, despite the fact that quarry waste, a material which was often considered to be inferior, had been used in the section of the work where the materials had been weigh-batched. The superiority of the weigh-batched concrete was shown by the great reduction in the standard deviations, which Mr Fisher had worked out as 690 lb. per square inch and 900 lb. per square inch for the continuous volume-batched concrete and for the batched concrete where the aggregate was measured by volume and the cement by weight, and 370 lb. per square inch for each of the two weigh-batched mixes. He did not know of any other large-scale work where such a small deviation of test results had been found.

Application of the laws of probability to the standard deviations recorded showed that the chances of any concrete failing to attain the specified minimum strength would be about 3-4 per cent. in the case of the continuous volume-batched mix, and 1 per cent. in the mix where the aggregates were measured by volume and the cement by weight, whereas the chances of failure with the weigh-batched mix were negligible.

Figs 27 showed that, from the continuous volume-batched mix, some 34 cubes out of 1,049 had failed to meet the specification; in the case of the mix where the aggregates were measured by volume and the cement...
by weight, 2 cubes out of 291 failed to meet the specification; no failures had been recorded with the weigh-batched mix.

Efforts to reduce the causes of variation in the quality of concrete were amply justified on the grounds that the client might well be in a position to specify a leaner and therefore cheaper mix, and also that the contractor's risk of being penalized for low-strength concrete was much reduced.

Were tests, in parallel with those of concrete on the site, made on the quality of the cement? Such tests gave some indication of the causes of the variations, and the information thus obtained would supplement that which had been given by Graham and Martin.1

Mr Fisher's own tests had shown that cements from individual works were reasonably consistent in quality but that there was a large variation between cements from different works. Had all the cement in the volume-batched mix been obtained from the same works or had a number of works supplied the cement? The cement used for the weigh-batched mix was all delivered in bulk from the same works, and he thought that that had contributed greatly to the reduced deviation. An analysis of the combined gradings of the aggregate given in Table 3, showed that very similar gradings had been attained. The most significant factor, as might be expected, was that the amount of material passing the 100-mesh sieve had been noticeably higher in the case of the weigh-batched mix than in the case of the volume-batched mix.

In general, the mix proportions—the ratio of cement to aggregate—were the same for the two mixes. The mix varied between 1:7 for a weigh-batched mix and 1:6.33 for a volume-batched mix. It was difficult to explain, therefore, the wide discrepancy between the average strength of the volume-batched mix and the mix in which the aggregates were batched by volume and the cement by weight—a discrepancy of about 800 lb. per square inch.

The presence of dust and the harshness of the sand-fraction in the weigh-batched mix would normally lead, as Dr Murdock had said, to the use of a higher water/cement ratio, but in the case of the weigh-batched mix a wetting agent had been added at the rate of 0.03 per cent. of the cement-weight, which tended to counteract the reduction in workability; in fact, all the average strengths, with the exception of one in the second mix, had been of the same order, and that would indicate that the water/cement ratios had been of the same order.

Mr Fisher thought that the time-lapse between concreting and striking shutters at Margam had been decided in a somewhat arbitrary way. Had any tests been carried out to determine what the time of striking the shutters should have been? Also, was the specified strength of the concrete in the floors 2,250 lb. per square inch? If so, two-thirds of the

specified strength—1,500 lb. per square inch—would seem to be a very low value. There was some reference to the time of striking shutters in the "Concrete Manual" of the United States Bureau of Reclamation, and it was recommended that the cylinder strength of concrete at the time of striking soffit shutters should be 2,000 lb. per square inch, which was equivalent to about 2,700 lb. per square inch in cubes. Since the economical use of formwork necessitated as rapid a turnover as possible, he thought that investigation on that point was urgently desired.

With reference to the wet-analysis of concrete, given in Appendix I, it had been assumed that the mix proportions were as specified. Should not the assumption be that the proportions of the various sizes of aggregate were as specified? Wet-analysis was, at best, a messy and tedious testing operation, involving such a number of assumptions, calculations, and corrections that the results could not be regarded as accurate. The wide variations which had been found in the strength of the volume-batched mixes indicated that there had probably been large discrepancies in the mix proportions from time to time. The possibility of detecting those changes by wet-analysis was very small.

Constant attention should be paid to continuous mixers in order to achieve results such as those at Abbey Works, which were, to judge from previous experience of that type of mixer, extremely uniform. Mr Fisher thought that a check on the correctness of the proportions was best carried out by careful observation of the through-put of each of the materials—aggregate, cement, and water—over a reasonable period of time.

Mr R. E. J. Worth referred to the 150 boreholes which had been taken on the 550-acre site. It had been a factor of primary importance that there should be no settlement of foundations, and the expenditure on boreholes had been, he thought, a small one. He had made a rough estimate from the figures given in the Paper, and it appeared that the proportion of the cost of the boreholes had been about one-sixth of 1 per cent. of the structural cost of the job.

The Authors had expressed the opinion (p. 9) that it was "unwise to specify that piles should be driven to a certain set. It is better that the piles should generally be driven to a minimum depth, dependent on the site investigation, and that they should then be capable of carrying a proof-load without exceeding a specified maximum settlement." Upon what basis had the Authors determined that minimum penetration? They had apparently discarded any dynamic considerations of measuring sets, and, in view of the very variable nature of the ground and the comparatively scattered distribution of boreholes, he would suggest that, if the minimum penetration were decided from tests on soil samples, it was likely to be inaccurate, as the subsoil varied rapidly from place to place. In the example quoted on p. 10, 82 per cent. of the piles test-loaded had satisfactorily conformed with the specification. Since settlement was so important a factor, did the Authors still consider that to be a
sufficiently large proportion? What was the order of settlement and the range of settlement of the remaining 18 per cent., which did not conform with the specification? Furthermore, was it not likely that, if 18 per cent. of the piles failed under the test load when tested singly, groups of piles carrying their full load might show a greater percentage failing under load?

The maximum length of time taken to test-load a pile had been about 12 hours—from the end of one day's shift to the beginning of the next; that appeared to be a very short time. Had the Authors carried out any longer tests to check the completeness of the settlement which they had achieved within that period of 12 hours?

The Authors had recommended (loc. cit.) that pile-testing should be started as soon as the pile-driving had started, and should proceed with it. Was that recommendation based on the experience on the site in question, or was it the procedure which had actually been carried out?

Mr B. F. Saurin, referring to the foundations, asked what types of piles had been used. The Paper did not show whether they were, in the general sense, toe-bearing piles or whether they relied mainly on friction. Since so many of them were in-situ piles, no doubt the question of the use of bulb-ends had arisen; did the piles have bulb or pointed ends?

The selection of the type of piles to be used had presumably received a great deal of attention before the work was started, but neither the Author nor Mr Atkins, in the associated Paper, had stated how the selection was made.

The forming of in-situ piles in ground containing thick peat-layers had doubtless have given rise to some anxiety as to whether the finished piles would really be satisfactory cylinders; and information gained from the experience on the site would be valuable.

Mr Douglas Weare asked whether the expanding aggregate, used in the packings to foundations, had proved to be an improvement over a closely compacted grout.

In connexion with the batch-mixing plant, he had noted the marked superiority of the weigh-batching method, and would welcome more information on the type of plant used, with some indication of the amounts of stored materials, cement, and aggregates, and of the segregation of aggregates prior to mixing.

With regard to the use of concrete-pumps, he had noted that at each of the six mixing-stations there had been a concrete pump of either 6 inches or 4 inches diameter. Had there been any differences in the behaviour of the 4-inch and the 6-inch pumps? Also, in view of the large number of cubes taken during the course of the work, had any opportunity been taken (and, if so, with what result) to check the effect of the concrete travelling.
through the various pipe-lines? Had any cubes been taken from the wet concrete-mix in the hopper immediately above the pump and, after allowing sufficient time for that particular mix to pass through the pipe-line, had any further cubes been taken from the wet mix as it was deposited? He had discussed that question with a number of engineers who had various opinions about the use of concrete pumps, and he would be grateful for any further information which the Authors could supply.

The Authors, in reply, stated that the columns had been set vertically and concreted in position before any girders had been landed on them; they had been, therefore, vertical at all temperatures before the girders had been landed. After the girders had been placed on top of the columns, they had been carefully measured and, since the standard tape was accurate at about 70° F., the tape and the girder had expanded together. The tape could, therefore, always be used to measure the length of the girder. When fixing the girder to the top of the column it had been necessary to strain the latter until its centre had met a mark on the girder; in that way, the design assumptions had been fulfilled on the site. The effect of temperature-change was apparent throughout the whole length of the building and, in the mould preparation shop, which was about 880 feet long, the expected temperature-range would cause a deflexion of a little less than 1 inch at the top of the end columns. (It should be noted that the deflexion of the tops of the columns was cumulative in each direction from the rigid bay, the position of which was clearly shown in Figs 16, Plate 3, of Mr Atkins's Paper. The effects of temperature-change on the columns were shown in Figs 18 of that Paper.) Shrinkage of the welds had had no accumulative effect, because each weld had been an independent operation.

The actual observed shrinkage had been of the order of $\frac{1}{2}$ inch at each of the main site butt-welds, and in the fabrication of the 110-foot girders in the shop itself the shrinkage had been about $\frac{5}{16}$ inch.

The welding had been tested satisfactorily. The operator could tell by experience, or by comparison with results which had been obtained before, the nature of the flaw in the weld.

The use of thermit welding for the butt joints in the gantry girders had been considered very carefully, but it had been thought that it would not be absolutely satisfactory. Thermit welding had been used, however, on the 375-lb. rails for the charging-cranes and also on the 375-lb. rails for the ingot- buggy; they were quite satisfactory.

A description of the methods of welding had already been published.¹

The 11-inch cavity-walls were quite separate from the rest of the structure. They were neither connected to, nor had support from, the steelwork. The columns were spaced at 40-foot centres, and in each length of 40 feet the brick wall was supported by three piers. The wall

was about 25 feet high and was capped by a concrete coping, to which a rubber jointing strip was attached. That strip was bent round and attached, at its other end, to the outside of the bottom flange of the gantry girder.

The vertical and horizontal loadings tests on the structure had been made only in order to compare the actual stresses with the calculated stresses, but the test-loads themselves were believed to have given stresses which would be normally encountered—of the order of 4–6 tons per square inch. The building had been designed for a combination of loadings which would very rarely occur—for example, maximum wind load and maximum surging with two or three cranes, etc.

The Authors were pleased to hear from Dr Murdock that the settlement under the brick-store was more or less what it should be.

The use of $\frac{3}{16}$-inch limestone for the concrete had caused an increased drying-shrinkage and liability to cracking, and in the early stages a few cracks had been noticed. It had become necessary to exercise more caution in its use. The cracking was probably caused by the settlement within the concrete. When using grit, bleeding had occurred and water had formed on the surface of the concrete. With limestone dust that effect was not so pronounced. Settlement had occurred in both cases, but the settlement of the limestone-dust concrete might have occurred more slowly, with the consequence that the concrete had been less plastic.

The Authors had carried out a number of tests on the expanding aggregate, using mixes of various slumps and sand-contents, and had not believed that it would be satisfactory; the test-results had been disappointing. It had, however, certain advantages. For instance, it held the mix together and prevented segregation, even when the mix was very wet; it was, therefore, easy to place, especially as a grout beneath large base-plates. On one occasion, when some mill machinery was being grouted up, two large plates had been placed alongside and grouted. Later removal of the plates had revealed an excellent result, although that might have had some connexion with the size of the plates.

Although expanding-aggregate concrete was easier to place in inaccessible positions than either mortar or dry small-aggregate concrete, that was offset by the high cost of the aggregate and the care required in handling it. In general, however, the earlier despondency about its efficacy had gradually diminished and, whilst the Authors remained unimpressed by its reputation, they were satisfied that it was better than a good cement-mortar.

The superiority of weigh-batching, from the point of view of the reduced coefficient of variation, was generally recognized, but it was necessary to weigh its advantages against the cost of the plant, the amount of work to be done, and so on.

The cement for the Port Talbot works had, in general, been obtained from one particular works for the continuous mixers and volume-batching,
and from another works for the weigh-batching. It was estimated that approximately one-half of the variation which had occurred in the weigh-batched concrete was caused by variation in the quality of the cement.

Mr Fisher had remarked on the greater average strength of concrete with volume-batched aggregate and weigh-cement as compared with concrete from continuous mixers. There was a tendency for the former concrete to be drier and richer than that from the continuous mixers.

The time for striking shutters had been determined by the contemporary demand for quick turn-round. The Authors believed that the use of cubes was quite reasonable for determining when the shutters should be struck, but they did not see the relevance of Mr Fisher's remark about the U.S. Bureau of Reclamation's recommendation that the strength of the concrete should be 2,000 lb. per square inch when shutters were struck. Surely the design stresses should be taken into consideration. It was true that, in the floors to which Mr Fisher had referred, the name of the mix had not been altered; it had still been referred to as a "2,250-mix" but, in fact, the strength of the concrete had been increased for the floors.

The Authors considered that the wet analysis of concrete provided a very useful quick test. The assumption regarding mix proportions related only to the correction factor and was of negligible importance.

Mr Worth had expressed the opinion that insufficient money had been spent on boreholes in the site investigation. However, the clients, the consultants, the main contractors and the soil-mechanics laboratory had, been satisfied that they had obtained sufficient information; they had been fortunate in obtaining it at so small a cost. Although the strata varied considerably, the building-site itself was not subject to great variation. The solid rock, or the harder material, was in the lower coal series and was overlain by the glacial deposit, on top of which were the alluvial deposits. It was in the last that most variation occurred. In the glacial deposit, the ground had become increasingly good. A large variation in the length of the piles had been anticipated, which would have been difficult to estimate accurately and would have made the use of precast piles uneconomical; that had been one reason for deciding on the use of in-situ piles. The boreholes had shown that the solid rock was fairly uniform and could be expected at depths between 20 and 50 feet, but there were variations in the higher layers and it had not been considered worth while to investigate them fully.

The Authors did not agree with Mr Worth that the specification for the pile-driving should have been more rigid. They considered it unreasonable to demand a certain carrying-capacity per pile and, at the same time, to state that each pile should be driven to a certain set. They considered that the skill and engineering judgement of the firms concerned were part of the services being paid for, and it was, therefore, quite sufficient to specify the load-carrying capacity of a pile. That would ensure that the foundations conformed to design leading. Any set of conditions which
necessitated a change in the specification could only arise from circumstances best dealt with on the site, as and when necessary.

The peat-layer had made it necessary to specify a minimum depth for the piles. For instance, at the south-west corner of the melting-shop, it would have been possible to attain a satisfactory set before the pile shoe had reached the peat. Extra boreholes had yielded the information that there was peat below and so the piles had had to be driven deeper.

It was not quite correct to say that 18 per cent. of the piles tested had failed; the ½ inch permanent settlement had not been specified as a criterion. There had not been an agreed specification in terms of so much settlement and so much load. The Authors had not wanted any piles to have more than ½ inch permanent settlement under the 50-per-cent. overload, and approximately 95 per cent. of the piles had shown permanent settlements of less than ½ inch. Only one or two piles had actually failed; efforts had been made to find out why they had failed, and extra piles had been driven where that had happened. That had been only a minor source of trouble, however; the major difficulty had been the liability that some failures might have passed undetected. It was probably true that the loading of a single pile would give a better result than the loading of a group of piles. Unfortunately, it had not been possible to load groups of piles; that would have required several hundred tons of kentledge. The Authors had been anxious to have routine testing, to ascertain whether the piles had reached a satisfactory stratum and to gauge the workmanship of the piles themselves; the pile-testing was also an encouragement to the pile manufacturers to make the piles well.

The pile-testing had been started fairly early, but not right at the beginning. The Authors' experiences indicated that the testing should have been started earlier.

The earlier pile tests had taken a long time—generally a week or ten days—but some of the piles had been tested quite quickly. So far as could be seen on the site, all the piles were equally satisfactory and, of course, it had been much more convenient to be able to spend less time on testing. Such tests had given the information required; rapid tests were more comparable with the actual loads that would come on the piles, because the maximum loads on the piles would generally occur with sudden loads, such as those on the cranes combined with sudden gusts of wind.

Piles with bulb-ends had been used almost exclusively, although there were some with pointed ends.

It had been considered that the piles should be toe-bearing and that friction should not be relied upon, because there would be a great deal of ground above the peat which would be liable to settle and possibly to cause an increased load on the pile.

In-situ piles had been chosen partly because of the irregularity of the site and the ease with which they could accommodate variations in length.
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The choice, however, depended chiefly on cost. The quantity of steel in
the pre-cast pile would have been greater than that in the in-situ pile.

Occasionally, excavations had had to be made around the piles, and
several piles in peat had been excavated. The results of the tests on
peaty water and the reaction on peat had shown that the only likely
occurrence was that the concrete might take longer to attain its strength.
Excavation had shown that the concrete in the piles was just as hard as
the concrete elsewhere and that the piles had been very well formed.

It had been necessary to stock-pile the gravel and stone, but not the
dune-sand. Aggregates had not, in general, been stock-piled at the mixer
set-ups, although, in the earlier stages, the double set-up, which was shown
in Figs 9, Plate 1, had had a fairly large area for stocking; it had been
intended to provide access by rail to that area. Storage space around
the mixers was generally limited and, at each set-up, was restricted to
about a day's output of concrete from that set-up.

Cubes had been taken both at the mixer and at the end of the pipe-
line, and no difference in strength had been found in them.

No difference had been found between the 4-inch pumps and the 6-inch
pumps with regard to the difficulty of handling, strength of concrete, or
workability. The 4-inch pumps had been introduced later, when a smaller
output had been required.

In the cases of the melting-shop and the mill, all the cement had been
stored in two large hangars, which had been placed back to back, giving
a storage-capacity of more than 1,000 tons.

The Chairman announced that the constitution of the Divisional
Board for Session 1950–51 would be as follows:—

Appointed by the Council in accordance with Rules 1(a) and 1(b) of
the "Objects, Constitution, and Rules":—

Chairman: Mr D. M. Watson
Mr H. F. Cronin
Mr G. P. Manning
Mr H. J. B. Manzoni
Mr J. C. Waddington
Mr P. G. Hudson *

Elected by the members present at the meeting, in accordance with
Rules 10 to 14:—

Mr H. J. B. Harding
Mr Savile Packshaw
Mr H. Shirley Smith
Mr W. Storey Wilson
Mr R. M. Wynne-Edwards

* Mr Hudson died on the 5th March, 1950, and the Council appointed Mr F. N. G.
Taylor to fill the vacancy.

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