co-operation with the operating staff, a siting committee making recommendations before the siting was decided upon.

In reply to Mr. Johnston's point about the staff training scheme, the author said he would certainly like to acknowledge the good work done by that scheme and that the signalmen were a vital factor in the system. The question had been raised as to whether modern systems were not just transferring the staff problem to another quarter by increasing the technician staff. In fact, very big staff savings were made by converting to the type of equipment described. To take the case where a man was held at a station all night just so that he would be there to perform one small operation once or perhaps twice in that period in crossing trains, conversion to a remote control system resulted in one technician looking after between 20 and 30 miles of track—say 20. With crossing stations every 5 miles on the average, there would be four on this section of track. To staff these for the tablet system on a five-day 40-hour week basis required four men at each station to provide continuous service, and therefore, without allowing for leave and sickness, 16 men were needed to keep that part of the line operational. With the remote control system one technician replaced anything from 16 to 20 men and 20 was the more probable figure.

In reply to Mr. Read's observation that the system could fail if the line had not been used for some time, the author said that the comment that occurred to him was "you use or you lose". With man or machine every faculty must be exercised or it disappeared. That was a most vital consideration with this type of equipment, and the principle applied no matter how well the equipment was designed. Presumably Mr. Read was referring to the rusting of the surface of the rails.

With regard to the suggestion that railway methods might be adapted to roads, the author pointed out that trains ran on rails and their presence on the rails could easily be detected, while that did not apply to road transport.

Mr. Holst had said: "You can't do business with a machine." In reply, the author said: "You can't do business without it either."

In reply to Mr. Christie's question as to what were the "special conditions" under which bells as well as flashing lights were installed at level crossings, the author stated that the policy was that flashing lights were provided primarily for motor traffic. "Special conditions" justifying bells were the use of a crossing by numbers of pedestrians, especially schoolchildren. Another factor was representations from various pressure groups.

In regard to the mileage of track under C.T.C. and what was proposed for the future, the answer was that the paper listed the systems working and under construction, and also that there were other sections under consideration.

The query about how the coding system worked was answered by an explanation that there were several systems using different principles. One system used a sequence of positive and negative impulses, another type used three line wires X, Y and Z, and each of these had series relays at each station, the code being made up of a sequence of X, Y and Z pulses. A third system, using only two line wires, had codes made up of a sequence of short and long pulses.

In replying to the questions concerning maintenance, the author referred to the "preventive maintenance" featured by Mr. Welch in his paper, and said that the aim in railway signalling was to have all maintenance "preventive maintenance". Regular overhaul after specified periods of service was laid down for all vital equipment. This period might be as great as ten years.

When a relay was overhauled, all important operating characteristics were shown on a sticker inside the relay with the initials of the relay mechanic responsible. The relay was sealed and was not permitted to be opened in the field. On the outside of the relay was another sticker with "Installed by ——" and "Date ——" on it. That fixed responsibility for the correctness of the outside connections, and recorded the age in service since last overhaul. Between routine overhauls each piece of equipment was regularly inspected, and if the contacts showed any sign of burning it was removed from service immediately. In this way it was possible to attain a very good record, and the relays on the railway system were making hundreds of thousands of operations every day. If a fault did occur, however, the piece of equipment was tested and the fault classified. Very few dangerous faults occurred.

CONFERENCE DISCUSSION

The Hutt Estuary Bridge

The paper, by W. G. Morrison, was published in Vol. 9, No. 11, pp. 356-75 (Nov., 1954), and described the design and construction of a bridge over the Hutt River, near its mouth. The bridge consists of five 105 ft. spans with approach spans. The width is 53 ft. 9 in. overall, with provision for a carriageway 30 ft. wide, a 6 ft. footway and a serviceway 11 ft. 6 in. wide for large water mains and other services. Special features to which attention was drawn are, first, the superstructure in prestressed concrete, secondly, the unusual foundation conditions, in that the piers are in effect bearing on a layer of marine clay overlying important artesian strata; and, thirdly, the administration, which was by "target" contract with selected contractors.

In introducing the paper, the author stated that levels taken two or three weeks previously showed that there had been a problematical settlement of one-eighth to one-quarter of an inch on the downstream end of the piers—problematical, because the levels did not close accurately enough for the result to be certain. There was practically no evidence of sag in the spans, an indication that creep effects had not made themselves evident in the previous two or three months. A slight opening had been found in the gap at the end of the beams, which would indicate that the compressive strain arising from the prestressed concrete had compressed the beams very slightly, about one-sixteenth of an inch or perhaps a little more, because the last measurements had been made in February, after a period of warm weather and the effect of expansion due to temperature would have closed the gaps. The gaps were slightly more open than in November. Those measurements indicated that there had been no alarming creep or sag. He had also learned that the flood of a few days previously had been about 50,000 cusecs, against the maximum recorded flood of 70,000 cusecs.

M. A. CRAVEN (Wellington) said that, while the author mentioned that the stresses were conservative, on closer examination they might be regarded as being more conservative than indicated in the report. The minimum strength called for was 5,100 lb./sq. in. at 28 days standard cured. To obtain that minimum an average strength of 6,000 lb./sq. in. was planned for. That meant that the range of compressive strengths at 28 days was from 5,100 lb./sq. in to 6,900 lb./sq. in. As concrete aged, so did its strength increase, and it would be reasonable to assume that, after approximately one year, when the bridge was subjected to full live load the concrete would have increased in strength by 25 per cent. of its 28-day strength. Thus, at an age of one year, there would be concrete strengths ranging from 6,400 to 8,600 lb./sq. in. He noted, at the same time, from the author's calculations, that the maximum stress under full live load would be 1,440 lb./sq. in. Magnel recommended that the ultimate strength of the concrete need be only three times the working stress, so that, if the working stress were 1,440 lb./sq. in., the minimum concrete strength need be only 4,350 lb. Instead, the minimum strength in the present case would be 6,400 lb., giving a factor of safety of nearly 4.5.

There would appear to be unnecessary strength in the main beams of the bridge after completion, even though all the early strength was required for the tensioning of wires. That might indicate that stressing should be done at a later age or, alternatively, that higher early strength concrete should be used.

Concrete Slump:

Originally, Mr. Craven said, he had suggested to Mr. Morrison that a two-inch slump with the usual tolerance be aimed at on the project, the reasons being: (1) Concrete of less slump was very difficult to compact thoroughly in moulds; good vibrating equipment was essential, as well as labour skilled in concrete placing, and it did not appear that both these commodities would be in plentiful supply. (2) Ready-mixed concrete was likely to be used, as there did not seem to be any likelihood of the concrete being manufactured on the site, because all those concerned with construction of the bridge were too busy worrying about other things; concrete of less slump than two inches would not be readily discharged from agitator trucks. The trend in England was towards the use of concrete mixes of the lower consistency range. Although such mixes could produce excellent results in the laboratory, observations in England indicated that they too frequently received inadequate compaction on the jobs. That led to conjecture as to whether the higher water-cement ratios of American practice were not preferable in the final result.

Unit Weight:

There was a standard test for determining the unit weight of concrete. He did not know whether the author had determined weight by that means or from measurement of test cylinders. Unless determinations were made by means of the standard test there was a severe danger of arriving at incorrect results.

Specifications:

The author had asked the question, "If the concrete delivered fails to come up to the guaranteed strength, what does one do about it after 28 days by way of penalty against the supplier?" That question presupposed that it was not possible to determine the quality of the concrete until 28 days after casting. If the concrete specification or the concrete testing did not permit an earlier decision, then it was indeed unfortunate and no answer could be suggested to the author's question. Normally, however, there was no good reason why the responsibility for any defective concrete could not be determined either immediately before the concrete was placed or within a few days after casting. In such circumstances there would appear to be little difficulty in having the defect put right.

Practically all testing could be done on the plastic concrete before it was placed in the formwork. Confirmatory tests could be made on hardened concrete cylinders, but the results of such tests really did not have anything more than confirmatory value, for, as the author suggested, little could be done towards effecting a remedy after the compression test results became available. If concrete was satisfactory in its plastic state before it was placed it should be satisfactory after being placed. If it was not, then those placing the concrete must accept the blame for improper placing. If concrete was unsatisfactory when plastic, it should not be placed.

It was a simple matter to determine the quality of plastic concrete by calculating its water-cement ratio. That gave a direct measure of compressive strength. Further, once the water-cement ratio of a mix was checked and a slump or unit weight test made, subsequent mixes could be readily checked merely by slump and unit weight determinations.

In this particular argument, Mr. Craven continued, it had been assumed that the concrete materials were up to specification. If the aggregate grading was not within specification limits, it would be immediately apparent in the various tests made, and also in the appearance of the concrete and its behaviour when being handled. It was also assumed that the cement was up to standard. If it was partially hydrated, due to damp storage, that would be visually discernible. If the cement was substandard in other respects, such faults might not be detected while the concrete was still plastic. Such occurrences, however, were very rare, much rarer than generally realized.

Cement Quality:

The author had suggested that low concrete strengths might be attributed to the quality of the cement. However, there was no evidence put forward indicating that the cement had been substandard; in fact, there was good evidence that there had been some other faults. In Fig. 17, a graph had been drawn of cylinder strength against unit weight. It was interesting to note that the suspicious cement, termed "last Blue Circle shipment", had a low unit weight (145.5 lb./cu. ft.), against a normal unit weight of 149.0 lb./cu. ft. The speaker said he did not see how the quality of the cement could affect the unit weight. Also, the low unit weight was in itself a clear indication that substandard compression tests would be obtained. In other words, if a unit weight determination had been made on the plastic concrete, it would have been realized immediately that something other than the cement was at fault before the concrete had been placed.

Yield:

The author asked: "How can yield be measured and what check is there on how much concrete remains in the truck after discharge?" The speaker said he could not see any difficulty in measuring the yield of concrete. There was a standard test for that, and his own experience had been that it was quite satisfactory. The speaker agreed with Mr. Cormack when he said, in the letter accompanying the author's paper, that beam moulds were unsatisfactory as a basis of measurement.

Agitator Trucks:

The suggestion had been made that concrete deteriorated in agitator trucks. That statement came as a surprise to the speaker. In the United States approximately 30 per cent. of cement manufactured was used in ready-mixed concrete, and on that account many tests had been made concerning the quality of concrete during and after transit. Such tests had shown that there was no deterioration within normal periods specified; in fact, the tests had in some cases shown that the quality was improved, due to the additional mixing.

The speaker concluded by congratulating the author on the very successful completion of the project and on the presentation of a paper which could be regarded as a most valuable contribution to the literature of the Institution.

H. R. BACH (Lower Hutt) said that his remarks would be grouped under three headings: (1) The background to the decision to construct the bridge; (2) Comment on some items in the paper; and (3) Prestressed concrete generally, and the present work as a major contribution to New Zealand engineering.

As to the background to the decision, the old bridge structure over the Hutt River Estuary, commonly known as the Hutt Pipe Bridge, was built shortly after the issue of an Authorizing Order in Council dated April 22, 1912. Its design was such that, even in a completely new condition, it was not suitable for carrying the loads that wished to use it in the period between 1930 and 1940, and its deterioration over those years was very much accelerated as the result of increased industrial and military traffic during the war. In 1940 the Wellington City engineer drew his council's attention to the further deterioration, and limits both as to loading and speed were maintained on use of the bridge. In 1943, the Engineer-in-Chief, Public Works Department, and the Wellington City engineer further reported on the matter, and in that year a conference of engineers representing the various local authorities interested in a proposed new Hutt Estuary bridge was convened by the Engineer-in-Chief, Public Works Department. Over the next few years, many proposals were considered and examined, including the possibility of strengthening the existing bridge, until, in 1951, a decision was made to proceed with the construction of the new bridge. The local bodies agreed that a commission should apportion the cost of the structure and the approaches, and Mr. Morrison was engaged by the combined local authorities to carry out the work of designing and constructing the bridge structure.

Commenting on some items in the paper, the speaker said that in Fig. 4, Superstructure, the cross-section of the bridge indicated that the carriageway width was 30 ft. He understood that there had been some criticism of this width for a two-lane bridge structure, but pointed out, with some emphasis, that it was easy to become confused between country highway standards and city street standards both in respect of bridge structures and roadways. Where there was always likely to be a considerable volume of cycle traffic, which it was not economically practicable to separate from vehicle traffic, a 30 ft. width of carriageway had been found very satisfactory. Over more than twenty years the Ewen Bridge at Lower Hutt had given satisfactory service for such mixed traffic with a 30 ft. carriageway. That was the standard adopted for most Lower Hutt bridges and for streets in which parking was prohibited. By double-line centre marking the bridge it was found, from many years' experience, that there was no danger of a three-lane flow of traffic developing.

Regarding section 2.1.5, dealing with frequency, would the author give a little more information as to what he considered an "undesirable frequency"?

Near the end of section 3.1, dealing with making the beams, it was stated that the cables were grouted with a mix of 10 shovels of sand, 1 cwt. of cement and 6 gallons of water. He complimented the author on his persistence in using sand, as, owing to difficulties arising, it had often been omitted, the probable result being a poorer job. Were the shovelfuls measured heaped or level? The use of such a measure with a cement mixture was, he believed, bad practice.

The meaning of the last three sentences of the first paragraph in section 3.4, dealing with transverse prestress, was not at all clear. Could the author re-examine the explanation so as to have the method correctly recorded?

In referring to the risk of loss of water from the artesian strata in section 3.6, reference was made to such loss affecting some 70,000 people. Actually, the figure would be nearer 200,000 people, as the City of Wellington placed considerable reliance at certain times of the year on the artesian supply.

As to section 4.4, dealing with supply and control, he appreciated the contention that slump was not critical when satisfactory steps were taken to ensure that density and cement content remained high, but was strongly of opinion that, for psychological reasons alone, a greater proportion of slump tests should be taken than 63 in approximately 500 or 600 loads of concrete.

As to the contribution the work had been to New Zealand engineering, most civil engineers now recognized the many advantages of prestressed concrete over ordinary reinforced concrete, especially for members subject to tensile or high bending stresses. Many statements had been made concerning the age of prestressed concrete practice, but when it was realized that the first full-scale practical application made by M. Freyssinet was in 1934, and that his cone-anchorage was not invented until 1939, and when allowance was made for the lack of research facilities in New Zealand, it must be considered very satisfactory that within ten years New Zealand was prepared to embark upon an extensive use of the new technique.

The circumstances leading to the employing authority agreeing to a prestressed design were unique. It might be accepted that a municipal engineer held in high regard by his council should be able to persuade the council to accept the new process. But in the present case there were Petone and Lower Hutt, which seldom agreed about anything, with two other Hutt Valley local authorities, sitting at the same table with Wellington City representatives. A consulting engineer, practically unknown to the local body representatives, was engaged for the work, and such was his faith that the new type of construction was the correct one for the work, he finally received unanimous approval for the project. The speaker paid a high tribute to the author, not only as being a skilled technical man who had to overcome many difficulties in what proved to be a job of more than usual complexity, but also as a firm but tactful negotiator who achieved success largely because of his inherent willingness to put all his cards on the table.

Concluding, Mr. Bach asked members not to confuse the two main parts of the paper. One was a description of a special type of structure, a prestressed structure, and the other of normal pier construction, but involving many difficult features. There was a tendency to confuse those two items in the matter of costs.

F. D. GRANT (Christchurch) said that, as one who had looked into some of the practical aspects of prestressed concrete in England recently and who had some similar work in hand, the paper had been of great interest to him. As a result of his experience, would the author consider the construction of long, heavy spans generally economical in prestressed work? Handling and lifting costs appeared to be considerable. In the Northam bridge in England, the holes for cross-stressing the various girder-deck members together did not register too well. That also appeared to be the case in the Hutt bridge. English engineers at the Northam bridge did not think that serious, treating it only as a cross-tie. What was the author's opinion?

F. W. FURKERT AWARD
Report of Judges, 1955

We have to report that in terms of the late F. W. Furkert's memorandum setting out the conditions of his bequest, we have carefully examined the relevant papers presented for discussion at the annual conferences from 1951 to 1955, both years inclusive.

After careful consideration of these papers, we are of the opinion that the F. W. Furkert Award for the period 1951 to 1955 should be given to W. G. Morrison, O.B.E., E.D., M.I.C.E., M.Am.Soc.C.E. (Member), for his paper "The Hutt Estuary Bridge".

The committee has, of course, adopted the policy of the Council that any paper which has secured any other award should not be eligible for an F. W. Furkert Award.

Yours faithfully,

D. S. G. Marchbanks
W. L. Newnham
C. W. O. Turner

Concrete manufacturing contractors in England favoured pretensioned and particularly straight-line beams, and concreting around the stressed wires, even in the case of the 105 ft. spans of the Northam bridge, where the contractors pretensioned the high-tensile cables, putting the necessary angles in the tensioning wires by the use of union screws. What was the author's opinion of that procedure, and would he consider stressing wires through precast holes and subsequent grouting preferable and really trouble-free?

The new B.E.A. repair shop under construction at London Airport was wholly a prestressed job, embodying many varieties of prestressed work, and was a beautiful job with excellent concrete and finish. Air Department engineers, however, were of the opinion that prestressing was economic on only portions of the work. The B.O.A.C. workshops, almost identical in plan, were being constructed in reinforced concrete, and the engineers concerned considered that that job, naturally heavier and more rugged, would probably be a million pounds cheaper. The speaker said that his overall impression was that for normal bridge work the use of pretensioned straight-line girders of readily handleable size in spans of about 25 to 35 ft. would be economic. What opinion had the author to offer on that point?

E. D. KALAUGHER (Lower Hutt) congratulated the author on inaugurating prestressed concrete in New Zealand, and complimented him on upholding the principles of the profession by making his experience available to others through the records of the Institution. The application of prestressing principles to bridge construction appeared to have reached its highest level on the Continent, where its development in competition with older methods of construction had been largely due to tenders being based on competitive designs. His own opinion was that successful prestressed construction depended largely on economic studies of handling methods, a function normally of the contractor. To that extent the speaker suggested that more important economies would be gained by designing to suit construction methods than by refining the structural elements of the bridge. Economic and efficient bridges could also be built in reinforced concrete and steel, particularly if simply supported spans were used.

It seemed that a large part of the economy in using prestressed concrete was really due to the use of simply supported spans. Costs increased more with continuous structures, which entailed much more rigidity in construction planning.

The bridging of a river was always subject to delays arising from high water conditions, and the greater the flexibility in the construction programme permitted by the design the more efficiently could the work be carried out. An important factor to be borne in mind was the morale of the workmen on the bridge construction. If they could see progress they would get into the job and do it well; if there were continual delays, particularly with continuous structures, they became disheartened and costs rose alarmingly.

Construction in prestressed concrete should not hold any terrors for any engineer, provided he held to a few basic principles. Units should be made to a size that could be comfortably handled. The smallest amount of handling was required when a unit was cast in place; the maximum when they were cast and then had to be handled as a whole. The choice should lie between those limits, which left the matter wide open to the engineer. In any system of precast work greater accuracy in dimensions was required, but the work must be set out so that adequate tolerances could be used at every stage. When detailing the drawings, engineers could well adopt mechanical engineering practice and specify tolerances for dimensions, and then, by working from a large tolerance down to a small one, some idea of the basic standards of accuracy required in the parts of the structure would be obtained.

Experience on the Hutt Estuary Bridge had once again demonstrated the difficulties associated with the use of temporary sheetpiling for cofferdams. The author had brought that out clearly in the detailed costs. There was still a fair amount of use for sheet-piling when a cofferdam could be incorporated in the permanent structure. If the sheet-piling was driven through ground of any depth it was not possible to get it out. The casualty rate was about 99 per cent. The fact that two caissons were successfully sunk in the present case should not encourage others to use caissons too widely. In the present case experience was available for the construction of the first two cofferdams to show that the riverbed did not contain any large logs or boulders. Bad luck with caissons could easily build up costs much higher than for a sheet-piling cofferdam, and in extreme conditions it might not be possible to sink a caisson.

The author had been disappointed that adequate water was not available for floating the beams into place. The bridge was also designed so close to the water level that there was not room for substantial falsework on which to erect portions of the beams. The beams had to be cast as a whole and flotation was not available. They were too big to carry out by any other means. As an illustration of the difficulties encountered, the Wellington Harbour offered the use of a barge with a heavy pile extractor already rigged, but, although the bridge was only a few hundred yards from Wellington Harbour, no way could be found of getting the barge to the bridge site.

The choice of the correct method of constructing foundations remained largely an art. Some assistance could be obtained from foundation explorations, where those were carried out, but the best of investigations could be very misleading. The English Institution's conditions of contract offered supervising engineers very much more latitude in that respect. In New Zealand there was a general tendency, even amongst contractors, to underestimate the cost of foundation work. It would generally be found that overseas costs were much higher than in New Zealand for similar classes of such work. The author's comments in introducing the paper suggested that he expected to find sag in the beams, owing to creep. The speaker said he presumed Mr. Morrison meant creep in the steel. The effect of creep in the concrete would be to "hog" the beams. Another reason no particular sag should arise was that, particularly in a highway bridge, the duration of the live load was small compared with the total number of hours the bridge was alive.

The age for stressing concrete was also important. In the early days of stressing concrete engineers were very conservative and waited 28 days before putting on a stressing load. Overseas practice nowadays was to stress the concrete within a week of manufacture, or as soon as the concrete obtained some testing strength. Could the author give a lead on that?

Mr. Kalaugher congratulated the author on his presentation of detailed costs. The author had obviously been constrained to stick to the mathematical principle that the cost of the superstructure should equal the cost of the substructure. That equation certainly held true for an infinite number of spans, but did it also hold true for a small finite number of spans?

CAPT. A. J. WATT, speaking on behalf of the Soil Bureau, D.S.I.R., said he had read with special interest those parts of the paper dealing with the excavation inside the cofferdams. Regarding the failure of the clay layer beneath the excavation, the author had assumed that the clay failed in direct shear on a vertical plane under the line of sheet-piling. The author's figures showed the factor of safety against a failure of that type was about 5, this figure being based upon the results of triaxial compression tests intended to indicate the bearing capacity of the material. It was thus inconceivable that a direct shear failure would take place unless the mean shear strength of the clay along the failure surface dropped to three or four pounds per square inch. Such a reduction in shear strength was unlikely to have been caused by any normal construction operations. Although for that particular clay the ratio of undisturbed to completely remoulded strength was at least seven, the sheet-piles were unlikely to have penetrated far enough into the clay for any appreciable remoulding to have taken place. It was possible, however, that some small disturbance under the piles, together with the extremely fissured nature of the clay, may have allowed local piping to commence. Even a small flow of water towards the base of the sheet-pile wall might have led to rapid and progressive softening of the clay.

Although it was possible that the clay was weakened in that way, it was very likely that the failure was not a shear failure at all. In a large excavation where a relatively thin layer of clay resisted a considerable upwards pressure, the clay might perhaps be considered as a slab resisting a uniformly distributed load. Whether the problem should be analysed using conventional structural formulae was doubtful. However, an attempt had been made to calculate the bending moment which might have occurred in the clay layer, assuming it was analogous to a uniform slab rigidly clamped around its perimeter.

The further assumption had been made that the clay had a finite tensile strength. It had been calculated that the clay layer would have required a tensile strength of 3 lb./sq. in. to resist the out-of-balance pressure to which the layer was subjected at the time it failed.

Recent laboratory tests on an undisturbed sample from the same stratum within thirty chains of the bridge indicated that the tensile strength of that material in a horizontal direction was only 2 lb./sq. in. It should be mentioned, however, that it might be dangerous to assume that a fissured clay of that nature had any tensile strength at all. The speaker said he understood it was standard practice in such a problem to ignore the strength of the material and to consider only the balance of pressures—*i.e.*, any excavation over an artesian stratum might be dangerous if the upwards pressure due to the artesian head exceeded the total weight of the remaining overburden. It would be normal practice to reduce the uplift pressure by pumping from the artesian bed, but in the present case, of course, the very large flow through the artesian bed made pumping impossible.

P. J. ALLEY (Christchurch) congratulated the author on his paper, particularly as it was the first time that the speaker had seen in the Institution's proceedings a paper on bridge construction in which reference was made to soils. Some soil mechanics figures had been given, and were very revealing, but if the author had gone further and given details of, say, sand, silt and clay content, and the usual soil constants below the foundations, they would have been very valuable, and would have shown how impervious the stratum was. Even though a stratum was impervious, it still conducted pressures. It could be a case of a gravel stratum below a clay stratum, and charged with water, the pressure being continued through the clay stratum, and reacting on the cofferdam and making it lift. The solution in that case would be to drive the sheet-piling further down, or put a filter in to weight it down. In the paper the preconsolidation pressure was given as starting at 1.5 tons/sq. ft. and finishing at 2.54 tons/sq. ft. Analysis of the consolidation curve in Fig. 2 showed that the pressures started at 2.54 tons./sq. ft., and finished at a higher figure. What was the author's interpretation of those figures? The pressure voids ratio curve showed a consolidation pressure equal to the overburden on the stratum. Was the use of the lower value justified, unless, as pointed out in the paper, there was an additional overburden in the past, or there had been some drying out in the stratum? It was unlikely that in artesian conditions there could ever be drying out of the stratum; there had probably been some heavy overload which had consolidated the stratum up to that point. The author gave a figure of one-eighth of an inch for settlement, whereas 2.54 in. had been predicted. The speaker suggested that the 2.54 in. of settlement could have taken place during construction. In most cases settlement took place very quickly at the start, and thereafter fell off rapidly.

J. H. WILLIAMSON (Christchurch) said that he had perused with interest the cost data given in the paper, but had not been able to make much of it. He took it that the costs set out in Table IV, showing the cost of the superstructure, were the estimating costs, and not the final costs. The amount allowed for insurances, and holiday pay, 8%, seemed very low, as holiday pay normally would be 4%, and the compensation rate for bridgebuilding 5%, a total of 9%. In addition, there were the odd holidays during the year to be taken into account. The Public Works allowance under that heading, 12½%, would appear to be nearer the mark, which would mean a difference of £1,300. Table VIII gave a final figure for total cost of £243,267, but in a note below additional amounts of £5,804 and £7,200 were provided for contractors' site supervision and office overhead. If that was all the contractors drew, it showed a very modest approach on their part, only £13,000 on a quarter of a million pound job. Regarding difference between the calculated volume of premixed concrete and the actual volume put in, from his own limited experience of using premixed concrete he thought 10% was the minimum allowance for wastage. It might be that some concrete stayed in the truck. Certainly some was wasted. But, even allowing a lot for wastage, it still did not account for the whole difference. He would appreciate any further information on that point.

L. M. McLEOD (Wellington) congratulated Mr. Morrison on an excellent paper and on the manner in which the paper had been presented.

In Table I, page 371, the overall cost of the bridge per square foot, excluding the cost of approaches and the design charges, had been given at £8.6, whereas it was understood that the final cost was in the vicinity of £11.5 per square foot. That appeared to establish a new high-level peak in bridge construction costs when one considered that the present average New Zealand cost of reinforced concrete bridges of comparable size on piles was £4.05 per square foot.

While it was appreciated that the foundations of the Hutt Estuary Bridge presented special difficulties, engineers might well ponder long over other forms of construction before committing the interests they served to prestressed concrete bridge construction.

The total surface area of the bridge paved with asphaltic hot-mix was 2,340 sq. yd. In Table VIII, page 374, the cost of the paving had been given at £5,000. The speaker did not know whether that was a final cost, but, even if it were so, the cost worked out at £2 2s. 9d. per square yard. On recent highway contracts, machine-laid hot-mix had been averaging £5 10s. per ton in position, 4s. 9d. per sq. yd. per in. of compacted depth.

Could Mr. Morrison give the cost per ton in position of the asphaltic mix used on the Hutt Estuary Bridge, and also the average compacted depth of mix laid on the carriageway and on the footpath?

W. M. SUTHERLAND (Auckland) congratulated the author on doing a wonderful job and in having the courage to go ahead with the new project. Not many would realize that at the moment 32 prestressed concrete bridges had either been completed or were in final stages of erection in New Zealand, as well as prestressed industrial buildings and reservoirs.

In the previous two weeks, the speaker said, he had made an attempt to analyse the cost per square foot of the prestressed decks of 14 bridges. It was impossible to arrive at a standard unit cost, because each job had some feature that was different from the others. However, the analysis was based on contract prices, and included cost of design, supervision, kerbs, handrails, and in most cases the bitumen deck. In all except one case he had required to estimate the cost of design and overhead charges. The first bridge constructed by his organization worked out at a unit cost of 64s. 3d. a square foot, for 1,245 sq. ft., and the contractor had said that he "did not lose very much money". In the case of three more bridges, some contractors said they did not lose any money, and others said they did not make any, and the average cost worked out at 44s., 50s. and 48s. a square foot.

Analysing the cost of the Hutt Estuary Bridge on the same basis, from the data given by the author—and it must not be compared with a single-lane bridge, or just a single 100 ft. span, because there were extra kerbs and handrails, etc., the comparable cost was 92s. a square foot, which was not very much when one considered that it was the first major prestressed bridge in the country. The average cost throughout on the 14 bridges he had analysed was 50s. a square foot, which indicated that prestressed concrete was not going to be expensive as one might be led to believe. The cost of the Hutt Bridge was not excessive when one realized that the contractors had to start from scratch and discover by trial and error the best method of doing things.

One factor in making prestressed concrete an economic proposition in New Zealand was that the country was short of labour and even short of equipment at the moment. Both the designers and the contractors had to develop an entirely new technique, and they had to work together to keep costs down. As an example of what could be achieved in that direction, the tender for a railway overbridge at Huntly called just before Christmas, worked out at a cost of 39s. a square foot for the deck, simply because the beams had been standardized and the contractor knew what he was undertaking. He suggested that engineers should be very tolerant for the next two years about the cost of prestressed construction until New Zealand had developed her own technique to suit conditions here. Concluding, the speaker mentioned that 2,000 sq. ft. of pretensioned flooring had been put in place in an Auckland two-storeyed building in four hours, the span being 30 ft., to take a live load of 180 lb./sq. ft.

Prof. H. J. HOPKINS (Christchurch) congratulated the author on his paper and on the work it represented. It was not often that an engineer had the chance of being a pioneer. The author had seized the opportunity and in so doing made an important contribution to New Zealand engineering. He had written a paper which would enable other engineers to start where he had left off. By that means engineering was bound to progress. The author had been a pioneer at the time the work was undertaken, but others had found out since more about what he had done and about prestressed concrete in general.

The speaker said he was interested to note the extent of the transverse prestressing incorporated in the design. In laboratory tests at Canterbury College units had been cast side by side, and the behaviour of the complete structure examined. The transverse prestress had been progressively reduced and the overall monolithic structure taken as the criterion. It was found that, for a small structure, a-quarter of the span, one-half of the width, or twelve times the depth of either the slab or the unit was the maximum spacing possible, but, even more important, if one wished a bridge to behave as a whole there must be transverse prestress of about 200 lb./sq. in. in order to have sufficient friction or shear force between the units, so that individual movement was prevented. The author's figures for the diaphragm of 280 lb./sq. in. average and for the deck slab of 310 lb./sq. in. falling to 263 lb./sq. in. indicated that for a practical case he had a fairly economical arrangement of transverse prestress. But when one considered that the author had in fact obtained the right amount of transverse prestress for the bridge to act as a whole, should it not be considered that the bridge did act as a whole, and that, in determining the loading to be applied to each individual unit, the total load on the bridge should be taken and be divided by the number of units? In the present design the author, quite understandably, had taken an H20-S16 loading on three units. It appeared that the load could very well have been distributed over 4½ units, which would represent half the width of the roadway. Examining Fig. 5, one wondered exactly how one would get any benefit out of that, however, because in that figure the maximum stress was given as 2,210 lb./sq. in., not 1,440 lb./sq. in., as mentioned by Mr. Craven. Mr. Craven's statement in that respect had been a little unfair, as in many prestressed concrete bridges the maximum stress occurred during operations and not under load. However, looking at Fig. 5, one wondered exactly what was the basis of calculation, because, although prestressing was an admirable procedure, it did not have any great value in itself; its value was in enabling one to do other things. If, after prestressing, there was a stress of 2,210, and the subsequent maximum stress was 1,440, and the top fibre stress was never less than 380, one wondered why all the prestress, because it seemed that there was much more than was required. Adding 2,210 to 1,440, and dividing the result by 2, gave about 1,800, which appeared to be the maximum amount of prestress required on the present work. Moreover, if, in future, the benefit of distribution across the whole of the bridge was to be taken, it would be necessary to look very closely into the cross-section of the units, so that the dead load plus prestress did not play such a large part in the design as appeared to be the case in the present instance.

A. N. GRIGG (Lower Hutt) said that, as a member of the joint local body technical committee, he had, at first, been most envious of Mr. Morrison's appointment to design and supervise the work under discussion; but later, as constructional difficulties arose, he had been very content to remain in the background and offer what help he could. Associated with the construction of the bridge was a considerable amount of engineering work, including 22½ chains of roading, some 22,000 yards of filling, the extension of a major drainage culvert, the removal of a house, and the construction of a traffic rotary island. With others, he had been concerned at the rise in the cost of the bridge from the original estimate of £125,000 to £152,000 at the time of determining the contract, and to a final cost approaching £300,000. He considered the three major factors contributing to the high cost were the type of foundations, the design of the superstructure, and the type of contract.

Regarding the statement in the paper that the local authorities were averse to penetration of the artesian strata by pile foundations, actually pile foundations were discounted by the author largely on the score of cost, and he had differed on that point from the start. If consideration were given to the possible artesian effect, an examination of the cross-section of the waterway would indicate that the chances of an artesian blow up with piling were not very considerable. There was an extensive overburden on the artesian strata of consolidated material, and a brief calculation showed that there was at least 40 ft. of effective water head acting against the artesian pressure. In boring elsewhere in the Hutt Valley no great difficulty had been experienced with the welling up of water around encased bores. A driven pile was a very much more secure restriction against the infiltration of water than a bore tube. The author's statement in Section 2.1 of the paper, that the quantity of permanent materials in the bridge structure did not have a particularly significant effect on the cost of the structure, was a very general one and would, surely, apply only when considering various types of construction. When the type of construction had been determined, the quantity of material involved must have an important effect on the cost.

The speaker supported Prof. Hopkins' remarks considering the quantity of materials involved in the superstructure. It would seem that substantial reductions in the quantity of materials used, and therefore labour, would have been obtained with very little change in the method of construction.

As to the basis of contract for the work, he would be pleased to hear other opinions on how a big contract should be controlled. The contract in the present case was on a cost-plus basis, with a target cost and a bonus. The speaker submitted that it was evident, particularly in the earlier stages of the work, that careful consideration of construction procedure was not applied on the part of the contractor. Could the author comment on that point?

What was the author's opinion as to the durability of mild steel for the expansion rollers on each beam? Would they function as perfect rollers, and what were the author's comments on their accessibility for future maintenance.

The speaker mentioned that, on alternative tenders being called recently in Lower Hutt for a comparable bridge of five spans and a total length of 480 ft.; it was significant that, although 14 tenders were received, none were for reinforced concrete construction and a tender had been accepted for a prestressed concrete bridge.

F. M. H. HANSON (Wellington) said that in the 105 ft. spans of prestressed beams there would be considerable vibration. He noted that a low-grade bituminous concrete had been work, he would be pleased to hear other used for surfacing. Did the author anticipate that that surfacing would have a normal life under the considerable vibration? Did the author consider any other design loading than H20-S16, in view of the fact that the area was an industrial one where heavy equipment and machinery would be transported? There seemed to be some confusion as to the final cost of the bridge. Could the author assist in arriving at the actual figure? Some members had felt sorry for the author in the problems he had had to face in the present work, but the speaker could assure them that Mr. Morrison had enjoyed every moment of it. There was a strong case in such an instance for regarding the work as applied research of national interest, and for the nation contributing to the cost in such cases. Engineers should curb any thought they might have that the only form of bridge construction that was worth while was a prestressed design. It was not the answer to all New Zealand's bridging problems.

T. J. PALMER (Hamilton), referring to the cross-section, asked whether the footway could not have been located on top of the pipes, doing away with the footpath on the other side and thereby eliminating two beams throughout the length of the bridge. He noted that, in the formula used for frequency, frequency was regarded purely as a function of the deflection and that, if all the units were constructed the same, each unit, theoretically, should have the same frequency. He understood, however, that if several of the units were tied together transversely, the amount of deflection decreased considerably and out of proportion to the number of units. Was any difficulty experienced with vibration, and, if a man standing on a single unit could cause a substantial deflection, what happened with a heavy load on the bridge?

G. COOPER (Wellington) said that, as one associated with the design of the bridge, there were several points he wished to make which might not be known to members.

It was significant that the beam section finally adopted was substantially the same as that initially submitted by Stressed Concrete Design Ltd. to the author before the speaker set foot in England to undertake some of the final calculations under the direction of that firm. The beam section might well have been modified subsequently, but he personally had met with considerable opposition in proposing various modifications to the design to facilitate construction. For instance, the initial proposal submitted, showing the diaphragms and transverse prestressing cables, square to the main beams, would have resulted in considerable construction problems. On the basis of the speaker's experience on the Continent, he had submitted that construction problems would be less if the diaphragm units could be precast, and, in addition, if the transverse prestressing could be established parallel to the piers, so that, in effect, each beam was the same as another. There was considerable opposition to those proposals, mainly, he believed, because Stressed Concrete Design Ltd. had not constructed a bridge of that size nor incorporated precast diaphragms and transverse prestressing parallel to skew piers.

A further consideration dictating the size of the beam section, which had been criticized by some as being over-conservative, was that the footway was initially shown on the same side of the bridge as the services. Consequently, an applied lane loading tended to throw more loading on to the edge beams, and the edge beams were initially designed on that basis. As a result of subsequent delays in respect of certain proposals for modifications to the design, it was decided to put construction in hand, and further modifications became out of the question.

One feature of the beam section in which a considerable financial saving could have been effected was by a reduction in the width of the top flange to enable access to the underside of the bridge through the deck rather than having to construct temporary scaffolding under the bridge structure.

Stressed Concrete Design Ltd. were specialists in the Magnel system of prestressing. The speaker felt that the transverse prestressing could have been better achieved by using the Lee-McCall or the Freyssinet systems, but that was a further point on which he had met with some opposition. Mr. Morrison had made it clear that the considerations were mainly political rather than technical, and at one stage the proposal was almost transferred to another designing institution altogether on that account. It was significant that, even during construction, Mr. Morrison had seen fit to change the upper transverse prestressing through the beams to the Lee-McCall system. The speaker believed even at the present stage that the transverse prestressing through the diaphragms could have been more easily and more economically achieved by the Freyssinet system as opposed to the Magnel system.

Mr. Cooper supported Mr. Grigg in saying that thorough preconstruction planning was very necessary to such work. Even during construction a considerable time was spent in the preparation of detailed drawings for the beam-handling equipment. No similar job could be fully planned before construction until the methods of handling had been tried in the field. He agreed with previous speakers concerning the need for close co-operation between the consultant and the contractor. The present work had been mainly a problem of construction rather than of design.

One point about the handling of prestressed concrete members was that they must never be assumed to be a static load. At all phases it was obvious that they were a dynamic load, and provision must be made accordingly in the handling devices. Concluding, the speaker quoted the words of one of the contractor's foremen: "This was not just a job; it was a challenge."

E. G. S. POWELL (Christchurch) said he was somewhat puzzled by the suggestions that the superstructure had been costly. On the basis of Mr. Sutherland's figures, he had plotted a graph with the abscissa distance proportional to the square of the span and the ordinate proportional to the cost per square foot of superstructure. The result had been a tolerably straight line with the Hutt Bridge cost figure just a little above. He then assumed the cost of the superstructure of the bridge as being directly proportional to the span, and drew another graph with similar results. When one considered that the Hutt Bridge was designed for a very much heavier loading than the others, then its cost compared favourably.

I. L. HOLMES (Christchurch) said that the paper would undoubtedly become a classic, but it would be a pity if, at the same time, it became a standard for other engineers in respect of working stresses. For the ultimate strength of the steel used, two working stresses were given—one at 120,000 lb./sq. in., and the other, the transverse stressing, 114,000 lb./sq. in. He wondered whether the steel stresses had not also been conservative, as had been noted in relation to the concrete stresses. As to the comment of an earlier speaker that prestressing had been practised only since 1939, the statement had been made at the jubilee of M. Freyssinet in Paris in 1954, that since 1939, under M. Freyssinet's direction alone, two-thirds of a million cubic yards of concrete had been prestressed—that was apart from prestressing done under the direction of M. Magnel and others in Germany and America. On a rough estimate, the Hutt Estuary Bridge comprised about 1/250th of that total.

Mr. MORRISON, in replying to the discussion, noted that no one had asked any questions about scour, or challenged the figures for safe scour depth. To him that was partly a relief, but also a little curious, as he had expected questions on scour. Nor had any questions been asked about the handrails. The detail of the handrails had not been stressed in the design, but if members examined the method of construction and design they would see that the handrails were a problem on their own.

He agreed with members that he had been fortunate in being able to persuade five local bodies to authorize a new type of construction. Admittedly his arguments to them were somewhat ill-founded as to the anticipated cost, but they had stood thoroughly on side and remained keen to see the job through.

Regarding Mr. Craven's contribution to the discussion, the speaker said he was not shaken in the slightest by Mr. Craven's comment on the conservative stresses in the concrete. Whatever was designed for was not always obtained. He had had too much to do with concrete over many years not to be thoroughly concerned at the possibility of having places where there was no concrete at all. One always liked to have a little up one's sleeve, no matter what tests were made of plastic or hardened concrete. Several times he had gone through the anxiety of working out what the effect would be at the bottom flange at the moment of stress as a result of perhaps grouting leaking out, or something of that nature. Where there was a plain gap one could see that the concrete had not gone in, but one wondered sometimes whether there were cavities in the web that one could not see. The first beam stripped showed cracks between the flange and the web which looked alarmingly like shrinkage cracks, and the speaker wondered what the effect would be on shear if the cracks went right in. However, he did not think they were shrinkage cracks, but due to the web not being vibrated sufficiently and settling down after the top flange was poured. Quite a number of things could happen. The men on the mixers and those on placing were not as conscientious as they used to be, and he firmly believed that the man placing the concrete had much to do with the finished job. Further, although one could not simply throw material away, another two inches of concrete added to the width of the bottom flange in every beam would require another 60 cu. yd. of concrete on the job, which, in terms of cost was fairly trifling.

As to determining the quality of concrete by unit weight, the author quite liked that method. Mr. Craven had advised using it on the present job, but the speaker had chosen not to accept that advice. Mr. Cormack, on a previous occasion, had stated that the unit weight method was not reliable or useful. However, it would be used on the speaker's next large undertaking, at New Plymouth.

The discussion on concrete caused the speaker to believe it was almost possible to say that one could make good concrete, but one could not prove how good it was. The variation in cements had been a cause of embarrassment on the present work. There were marked differences in performances with different cement. The speaker did not see how one could tell just how good concrete was in its plastic state, without the most elaborate laboratory check. It seemed to him that the final strength was something that could not be ascertained until the hardened concrete was tested. He would go some distance with Mr. Craven by saying that Fig. 16 showed that, in respect of the poor cement, there must have been some fault in grading and density, which made the speaker lean towards unit weight as a ready and early means of measuring the quality of the concrete. One was entitled to assume that if Certified Concrete Ltd. could not produce concrete of a better or more consistent quality, with all the means at that firm's disposal, then no one else could.

Mr. Morrison said that he would refuse to "yield ground on the question of yield". A steel mould of 20 cu. yd. capacity bulk measurement was used, and if 20 cu. yd. of concrete were bought that 20 cu. yd. should fill the mould. If it would not fill the mould, then those who sold the concrete should explain how much of the mould the concrete would fill when they sold it. There might be differences of opinion, but the speaker thought the customer should be told that 10 per cent. allowance should be made for waste.

Mr. Morrison thanked Mr. Bach for his contribution to the discussion, and also for the co-operation he had extended all through the work.

Mr. Morrison said there were several questions raised in the discussion on which he would like to submit written comment later.

Regarding the question of cost, the cost of all items in the Hutt Bridge had always seemed higher than the speaker thought they should be. On the other hand, he had since persuaded the Eketahuna County Council to set up a factory to make 30 ft. beams of rectangular cross-section, using the Lee-McCall system, and everything there was working out cheaper than he thought it would, a principal factor being that the work was being undertaken by county workmen as undercover work, and another that advantage was taken of the Hutt Bridge experience and a method of handling worked out for a minimum number of men.

An important factor in such an undertaking as the Hutt Estuary Bridge was to have the right men on the job, men who were not scientists, but engineers. In Mr. Kalaugher, for the contractors, and Mr. Cooper associated with the speaker, a combination was created which produced quite a lot of talent.

As to accuracy of dimensions in precasting, that was not so simple to achieve as it might seem.

Regarding comments on the cofferdam, it was very interesting to be able to go down and see the position for oneself, but he sometimes thought that often too big a penalty was paid for "being able to go down and see". The more he thought about cofferdams the less he liked them. Every endeavour was made to extract the piles with as good machinery as could be obtained. Unfortunately, there was not much heavy construction plant in New Zealand.

As to creep in the concrete making the beams "hog" or sag, he suggested that members think about it—it was very interesting.

Regarding Mr. Alley's and Captain Watt's comments, he was still puzzled as to why the cofferdam itself came up. If the bottom "blew", then that was an ordinary construction hazard, but how did the clay or other stratum behave so as to force the cofferdam itself up? He could not agree with Captain Watt that such a situation was a standard case for dewatering. He had studied the theory of the creep of water, and the ratio of the creep path to the head, but he still did not see exactly how the clay failed so as to lift the cofferdam as well. If the bottom "blew", then the explanation was simple enough.

The speaker explained that, from the commencement, time had been the essence of the undertaking. Looking back now, and having had two or three years to think about it, he thought that closer investigation of the clay might have been made. He hoped to provide more information on that matter, and he was sure Captain Watt would assist by adding something in writing on the soil mechanics aspect.

Regarding Mr. Williamson's comments on costs, section (c) in Table IV showed the actual cost of the five 105 ft. spans, whereas sections (a) and (b) gave estimated costs. Mr. Williamson was quite right as to the holiday percentage being light, but that was what was accepted by the contractors. The speaker also agreed with Mr. Williamson that the contractors did not get very much of a financial return from the undertaking, but they were keen to try something new and to gain the experience. At the conclusion of the project, however, the joint committee of local body engineers decided to make a gesture to increase the contractors' emolument.

Mr. Sutherland had asked some interesting questions about costs per square foot, but Mr. Powell had partly answered them. The cost per square foot must increase with the span; and yet, if the span was very small, then the bridge was mostly abutments and the cost rose again. There were always different features on each undertaking which made a fair comparison difficult. However, £4 a square foot for common types of bridge in concrete was not too usual nowadays. The *Concrete Year Book* for 1954 gave the cost of various prestressed bridges in England, including the Stepney Bridge, £6.8/sq. ft., and the Hackney Bridge, £6.2/sq. ft., for the deck only.

Regarding Mr. Sutherland's remarks, Mr. Sutherland had been through the mill and had a fellow feeling on the subject. Table III showed that the cost of prestressing labour once full production was attained was about half that for the overall period, which meant much less than half the cost of the first few beams. Mr. Morrison said he was more and more convinced that considerable economies could be effected by designing to suit favourable construction methods, having regard to the resources, plant and skilled labour available.

As to Professor Hopkins' remarks, the design for a number of parallel beams such as was adopted did introduce uncertainties as to the distribution of load. Model tests were probably the only practical way of finding out just what happened. Regarding the distribution of the load over five beams instead of three, the speaker thought that what was set out in the paper as to the action of the transverse prestressing was fairly correct, although he admitted that he had written the analysis for the paper without having examined the matter before, as other methods presenting themselves appeared to be much too complicated. The bridge must deflect transversely as well as longitudinally and the deflection contours must follow some normal pattern. From that it was possible to gain a fair idea of how the transverse prestressing would act. He had been interested to notice in a recent paper by Mr. Donovan Lee that in railway bridges with two tracks the favourite solution now was to construct two bridges. The interaction of the wide members of a bridge under a concentrated load was most difficult to design for, and even if all the stresses were determined the effects of tension and racking could be very undesirable.

Professor Hopkins had hinted at what the author should have stated previously, that "prestressing" was not a proprietary article or a patented term; it was simply applying some stresses beforehand, under control. Many a structure was prestressed in some form or another—and must be so—without the application of any special system or calculations. In handling a prestressed job, however, and a precast job particularly, the engineer was compelled to realize that he was dealing with a live material, that deflections did occur and must be calculated beforehand. It was excellent discipline for the engineer and made him appreciate perhaps what he had not appreciated nearly so fully before.

Mr. Grigg's comments, as one would expect from his having been a close observer of the work at all stages, were highly informed. The speaker's broad reaction to Mr. Grigg's comments was there was no doubt that taking plenty of time for the major design did pay dividends. He agreed with Mr. Grigg's comment that the contractor did not plan the work well enough in the early stages, but that was because there was a succession of two or three men in charge on the site, none of whom had sufficient experience. Even when Mr. Kalaugher took charge, various things had already been decided which set a course of action perhaps not the best. Moreover, the arrangement had been that the designer should say what was to be built, and the contractor would say how it was to be built. As a result, the speaker was satisfied that the one who conceived the scheme, which included design and construction, could do so properly only if he could ensure that construction proceeded according to a proper plan.

As to the question of vibration raised by Mr. Hanson, the author said he did not know a great deal about that. Mr. Norman's formula had been quoted. At one stage the speaker had been concerned that the vibration period seemed to be rather an unfavourable one. The light standards on the deck had vibrated to an alarming extent, and he had assumed that the period of vibration of the standards was somehow in phase with the deck, so they had been filled with concrete to alter the frequency of vibration. That had seemed to settle it. There was a certain amount of vibration, but he did not consider it remarkable. There was, of course, a heavy dead load compared with the live load.

Regarding the bitumen surface, he had simply secured the best advice he could in that matter, from the Ministry of Works, the Highways Board and the Wellington City Council.

Mr. Morrison thanked Mr. Cooper for his comments. Mr. Cooper's contribution to the design and construction of the bridge had already been acknowledged. He echoed Mr. Cooper's comment that in one structure there might be a case for the use of the Magnel system, in another the Lee McCall, and in another the Freyssinet.

The CHAIRMAN thanked the author on behalf of all members for a valuable paper on a complex and difficult undertaking.

WRITTEN DISCUSSION

W. G. Morrison

Written Reply to Mr. Alley

Nature of Clay Stratum:

Captain Watt had kindly supplied figures for three samples of the clay. These showed:

	No. 1	No. 2	No. 3
Clay $<$ 0.002 mm.	26%	28%	22%
Silt 0.002—0.02 mm.	70%	68%	73%
Sand $>$ 0.02 mm.	4%	4%	5%

Settlement:

Though in his introduction he had said that the settlement of the bridge since it was opened had been very slight, it was stated in 3.6 of the paper that the settlement of Pier 3 had been $4\frac{1}{2}$ in. during construction. This was, of course, an exceptional case. The settlement of Pier 4 as given in the second paragraph of 3.6 should answer Mr. Alley's question. The caissons hardly settled at all, owing, presumably, to skin friction above the clay; but they might yet settle if a flood removed most of the material above the clay even temporarily.

He did not understand the question as to preconsolidation pressure. The initial pressure on the clay owing to overburden was a mean of 1.58 ton/sq. ft. The mean added load was 1.06 ton/sq. ft., giving a final pressure of 2.64 ton/sq. ft. The voids ratios had then been taken for these pressures per Fig. 2.

Concrete:

The concrete section of the paper, particularly with regard to Fig. 16, was not complete without the following data, kindly supplied by Certified Concrete Ltd.

Intrinsic weight:

¾ in. aggregate	166.7 lb./cu. ft.
Sand	159.1 lb./cu. ft.
Cement	194.7 lb./cu. ft.

* * *

Cofferdam Uplift

Whilst not being entirely convinced by Captain Watt's arguments during the formal discussion, he had had further consultations with Captain Watt, and between them they had formulated what he thought was the real nature of the failure of the clay under Pier No. 3. The unusual factor was that the sheet-piles of the cofferdam and the clay under the cofferdam were forced up as a unit. The surface of the sand and gravel inside the cofferdam remained practically level and the timber staging piles alongside the cofferdam (Fig. 11) did not move. Therefore the failure was very localized. There was no evidence of the escape of artesian water inside or outside the cofferdam. Thus he discounted the theory of piping and consequent softening of the clay.

However, Captain Watt had given a lead by his calculations and his tests, which indicated the tensile strength of the clay to be only 2 lb./sq. in. If the bending moment due to uplift on the clay were calculated on the simplest basis as a continuous slab at $wl^2/12$, the tension at the bottom of the clay layer directly under the sheet-piles (Fig. 11) would be 5 lb./sq. in. Then, if tensile failure took place, artesian pressure would effect an entry and by horizontal pressure would "prestress" the clay beam and also weaken the clay in shear under the sheet-piling.

At the same time it would be supposed that the cofferdam bracing had not been quite tight, so that as the cofferdam was pumped out the bottom ends of the sheet piles below the bracing would deflect slightly inwards. This would have the dual effect of (a) gripping the plug firmly enough to cause the piles to lift with the plug and (b) removing any friction or shear resistance to uplift in the clay outside the sheet-piles. They would in fact be lubricated by river water percolating downwards.

All the considerations outlined above would provide for the uplift of the plug intact with the sheet-piles as actually happened.

CONFERENCE DISCUSSION

Plastic Design of a Pitched Portal Roof

The paper, by I. L. Holmes, was published in Vol. 10, No. 1, pp. 2-10 (Jan., 1955), and described a roof structure designed by the plastic theory of J. F. Baker and others. The roof, which is of 67 ft. span, covers part of a garage. Main frames at 44 ft. 3 in. centres are regular pitched portals and take the sidesway thrust from irregular intermediate frames at 14 ft. 9 in. centres, as transferred through composite beams at eaves level on one side only. The justification for the plastic method of analysis was given, and comment was made on the consequences in the analysis of the mixed framing system. Full references were given for the design method.

Prof. H. J. HOPKINS (Christchurch) said that Professor Baker's general concept was a great challenge to engineers everywhere, and its presentation in Mr. Holmes's paper was a great challenge to New Zealand engineers. The speaker drew attention to the first sentence in Professor Pippard's statement quoted on page 9: "The intelligent and economical design of engineering structures demands knowledge of their behaviour under load. . . ." When Professor Baker first presented his paper to the Institution of Civil Engineers, Professor Pippard observed that two points were discussed by the author which were separate and distinct—first, the philosophy of structural design, and, second, a technique. The speaker wished to deal with the present paper along those lines. In February, 1950, Freyssinet stated to the Institution of Civil Engineers:

"So heavy was the intellectual oppression exercised by a handful of mathematicians obsessed with their science and blind to reality that an unquestioned belief in the use of Young's modulus for concrete was held, without a valid basis and in spite of many proofs to the contrary, by all our professors, and, indeed, by all technicians."

What the designer had been doing for many years was to pay a certain amount of attention to the properties of materials and practically no attention whatever to change of those properties by structural form. Steel, of course, was a consistent material, with well-known properties, and the yield point could always be forecast fairly accurately. The designer worked out the particular stresses in a structure in a way that would allow some sort of factor of safety on a well-known property of a material. Professor Baker had presented the idea that there should be a factor of safety on the structure—that was the important thing. With steel, it was interesting to note that due regard was paid to the properties of the material and practically none to the structural form. With concrete, attention was paid neither to the properties of the material nor to the properties of the structural form. He congratulated the Christchurch City Council on passing the present design in a well-known elastic material to allow it to be designed by a so-called plastic method. He hoped that, sooner or later, the Council would allow a plastic method of design for a plastic material.

As to the actual technique, he wished to amplify the parts of the paper dealing with elastic design in the particular structure. Mr. Holmes had quite rightly been concerned to present, in its full flower, the plastic design of the structure, and obviously had not wanted to go too much into the elastic design, which was well known to most engineers present. One point, however, should be made regarding Frame B, referred to on page 5. The frame there rested on a column on the right-hand side, and was constrained at ground level by, presumably, an immovable foundation. On the left-hand side it rested on a beam, which could deflect between Frame A and Frame C. In doing so it would relieve and reduce the horizontal thrust H which was set up. Mr. Holmes suggested, quite rightly, that in the plastic design one was not particularly concerned with individual distortions of the individual members; but in the elastic design one was, and if one was to compare the plastic with the elastic in the particular design, then it was reasonable to do an analysis of the elastic properties of the structure. Members would note that Beam E, framing into Frames A and C, produced a horizontal force at the knee of Frame A of 15,200 lb. That force—and the speaker said he had not included the dead load from above—when applied produced a deflection of about 2 in., to which was added the deflection on Beam E, spanning between Frames A and C. The complete result of both those deflections, going sideways, was to reduce the horizontal thrust set up in the frame to just over 6,000 lb., and in the plastic condition to just under 6,000 lb. It would be clear, therefore, that, what-