THE HUTT ESTUARY BRIDGE

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The paper describes 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 is 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. The bridge was constructed for the combined local bodies comprising the Lower Hutt City Corporation, the Wellington City Corporation, Petone Borough, Eastbourne Borough and Hutt County. A Government contribution to cost was made. The author acted as consulting engineer and was advised from time to time by the technical committee which was formed of the engineers to the combined local bodies and the Chief Designing Engineer, Ministry of Works. The chairman of the combined local bodies committee was P. Dowse, Mayor of Lower Hutt, whose close interest and generous support at all stages of the work is gratefully acknowledged. The contractors for the work were Messrs. Wilkins and Davies (Construction) Co. Ltd.

1. DESIGN: GENERAL

1.1. Waterway

THE figure of 100,000 cusecs was given as peak discharge for the Hutt River by the officers of the Hutt River Board. This figure is understood to be that on which stopbanks and other protective measures are designed. It was accepted without further investigation as a maximum discharge for design purposes. The maximum recorded discharge is less than 80,000 cusecs. The width of waterway was decided somewhat arbitrarily by analogy with existing bridges upstream as 525 ft. between main abutment centres, and the effects of scour on piers and abutments were considered in relation to this dimension.

1.2. Services

At first glance it would appear more economical to design a bridge in which the service pipes could be carried below the roadway without adding to the overall width of the bridge. Such an arrangement would, however, have made installation and maintenance of the service pipes very much more difficult, and would have required the carriageway to be at a higher level in order to give flood clearances. It was decided that the best way to carry the pipes would be in a separate serviceway so that they could be placed by crane from the carriageway.

1.3. Superstructure

Comparative designs were made for steel trusses, steel plate girders, reinforced concrete and prestressed concrete. Structural steel was not regarded as suitable, because of the severe exposure to southerly gales off the sea, nor did it appear to compare favourably in cost, particularly if maintenance painting was taken into account. At a comparatively early stage, therefore, attention was directed to the cost and other features of reinforced concrete compared with prestressed concrete.

Trial designs were made for seven 75 ft. spans, five 105 ft. spans and (in prestressed concrete only) for four 130 ft. spans. These showed the prestressed simply-supported spans to be more economical than continuous reinforced concrete spans. Simply-supported spans were preferred on account of probable subsidence of piers. Naturally the shorter spans showed economy in superstructure, but this had to be weighed against the cost of a greater number of piers. When the costs of both piers and superstructure were considered, economy appeared to favour prestressed spans of 105 ft., and these were adopted.

In the light of actual construction costs, it is possible that 130 ft. spans in prestressed concrete would have shown an economy. Certainly the saving of one pier would have paid for the extra steel and concrete in the longer spans, but the handling of beams 130 ft. long was regarded as ruling out this design. It was considered that 105 ft. beams were as long as could, for practical reasons, be made ashore and subsequently placed on the piers. Any *in situ* prestressed construction would have required a staging subject to flood risk.

An interesting suggestion was made by a London consultant that a prestressed box design be adopted, on the style of the Scalyn Bridge, with a central span of 230 ft. Such a design would have been very economical of concrete and steel, and would have had a very fine appearance. It was rejected because of flood risk to the necessary staging, the great concentration of weight on two piers, and, it must be admitted, the unprecedented nature of the construction in New Zealand. Had the foundations been on rock and the flood risk absent it would have been an admirable solution.

Prestressed, precast construction with 105 ft. spans was thus decided upon for the following reasons:

(a) Economy.
(b) Non-continuous spans to eliminate damage by settlement of piers.
(c) Speed of construction in that beams would be made while the piers were being constructed.
(d) Very low flood risk.
(e) Low maintenance cost.

1.4. Piers

Test borings taken near each abutment and on the site of each pier showed a very uniform layer of marine clay

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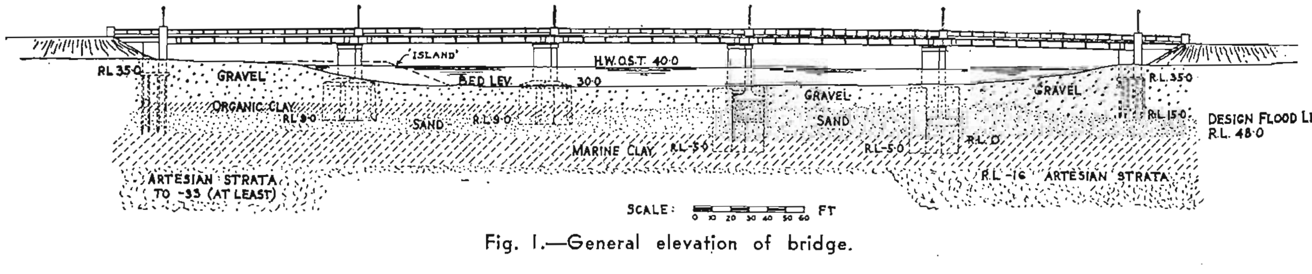


Fig. 1.—General elevation of bridge.

about 16 ft. thick at a depth which seemed to be well below the level of scour (see Fig. 1). Under the clay were layers of sand and gravel constituting the water-bearing artesian strata which are of vital importance to the whole of the Hutt Valley. Tests made by the Soil Bureau of the Department of Scientific and Industrial Research showed the clay to possess enough strength to carry the expected pier loads if suitably distributed, but to show very considerable reduction in strength when remoulded. Figure 2 shows the result of a consolidation test made by the Soil Bureau. The choice was then broadly one of founding the piers above the clay or of carrying them down to the much more reliable strata below the clay.

Trial designs were made of eight types of pier and the cost estimated. At this stage it must be admitted that the estimates were all low and that the comparisons which were made on the basis of cost might be in error.

However, the pier types under review were:

Above the clay:
(1) Solid pier on spread base.
(2) Hollow pier on box caisson.

Into the clay:
(3) Solid pier on concrete piles.
(4) Solid pier on timber piles.

Through the clay:
(5) Franki piles.
(6) *In situ* piles on grout pad.
(7) Three 6 ft. caissons per pier.
(8) Low piers on 75 ft. precast piles.

On receiving the report from the Soil Bureau as to the low shear values of the remoulded clay, it was thought that short piles driven into the clay would be of little value, so solutions (3) and (4) were rejected. The local bodies depending on artesian water supplies were most averse to penetration of the clay layer, and this was accepted as something to be avoided.

Particular atteniton was therefore given to the design of piers founded above the clay, and such were built, though not all to the original design, as will be described later.

1.5. Abutments

Settlement as forecast by the Soil Bureau, and also the lateral thrust of the approach filling, indicated the desirability of keeping the fill away from the abutments. These were designed to be carried on precast piles as a precaution against scour, though the strata for 30 ft. below the abutments is judged to be quite capable of carrying the loads imposed.

1.6. Approach Spans

Short approach spans of 30 ft. length were adopted at each end of the bridge, partly to accommodate settlement of the approach fill. These spans are constructed with partial hinges on the abutments so that they can rotate about

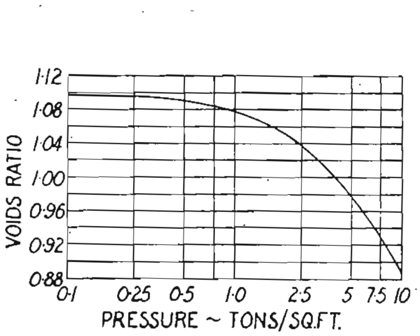


Fig. 2.—Clay consolidation.

these points, and are carried on seatings on the approach fill so designed that in the event of undue settlement the ends of the approach spans can be jacked up.

1.7. Profile

The required flood clearance is at R.L.50 and the bridge profile was designed as a parabola with a rise in the centre of 2 ft. This figure was selected so that appreciable differential settlement of the piers might take place without causing obvious sags in the bridge profile. The parabola adopted also conforms reasonably well with the "hogging" of the 105 ft. beams under prestress. Figure 1 shows a general elevation of the bridge.

1.8. Handrails

For æsthetic reasons the original handrail design was in open steel. Various types were considered, and a short length was made as a demonstration. At a later meeting the committee decided in favour of a solid-type wall on the south side of the bridge to give protection to pedestrians from the southerly gales. This was made 3 ft. 9 in. high. Similar solid concrete walls were designed on either side of the carriageway, but only 3 ft. 0 in. high.

1.9. Expansion Joints

After thorough discussion of a number of types, it was decided to adopt an open gap faced with $\frac{3}{4}$ in. galvanized steel plates (see Fig. 3).

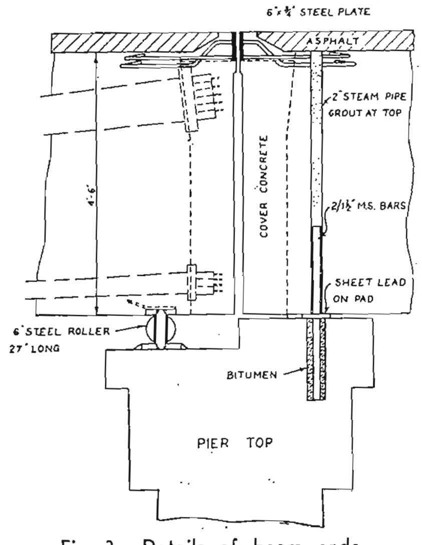


Fig. 3.—Details of beam ends.

1.10. Surfacing

Following various precedents overseas, the original design provided for the crossfall to be built up by means of a thin layer of screed concrete reinforced with mesh. The whole surface of the carriageway was then to be covered by $1\frac{1}{2}$ in. of plant mix. Second thoughts suggested that the screed concrete would be so thin near the kerbs that, if not perfectly bonded to the prestressed work, it might loosen and fracture under traffic. It was therefore decided at a meeting of the technical committee that the crossfall be built up in a coarse plant mix and that the surface be as originally designed—that is, of $1\frac{1}{2}$ in. of dense, impermeable plant mix. The footway also is surfaced with plant mix, while the serviceway has been sprayed with a primer and covered with fine grit.

2. DESIGN: DETAILED

2.1. Superstructure

For any conventional reinforced concrete bridge the design is worked out according to well-defined rules and specifications and may proceed with little regard to the method of construction. The method of construction is usually, and quite properly, left to the contractor, who is guided by stated requirements in respect of a few points such as deflection of formwork and of concrete strength when formwork may be struck. In a prestressed bridge, however, and particularly if important members are precast away from their final position, the design must take into account every detail of construction. The merit of the design cannot be judged apart from the question of cost, and the cost will depend on many factors other than the quantity of concrete or steel. It is necessary to consider how many units must be cast; where and in what sequence they shall be cast; how and where they are to be assembled and stressed; how they are to be transported to their place on the bridge, and what transverse prestressing is necessary. In respect of precasting it is by no means easy to ensure the accuracy required. In respect of all the foregoing the sequence and co-ordination of operations and the degree to which both labour and plant can be efficiently employed are likely to have a far greater effect on cost than a substantial variation in permanent materials. This comment applies to bridge construction generally, as is shown in Table II, but it applies with particular emphasis to precast, prestressed construction. In short, thorough planning of every detail and process is essential.

Once a decision had been made to adopt prestressed construction on 105 ft. spans, the next question was the size and type of the beams, and the system of prestressing. The Magnel system was chosen on the grounds that the anchorages seemed to be the most positive, even though they might be among the most expensive. The fact of stressing wires in pairs instead of in larger groups appeared to ensure more uniform stressing and less likelihood of the breakage of a wire being overlooked. Correspondence was entered into with Stressed Concrete Design Ltd., London, as to a suitable design, and preliminary drawings were received for an I-beam design and for an alternative in U-beams with an *in situ* deck. The writer was not satisfied with the measures proposed for dealing with longitudinal shear between the U-beams and the *in situ* deck, nor did the proposals for grouting the cables seem adequate. Bridges of this design have, however, been built. Both designs were carefully examined from the production aspect, and the U-beam design seemed to require a very large casting yard with a 5-ton crane in constant attendance. For these and other reasons the I-beam design was accepted (see Fig. 4 of superstructure).

At about this stage G. Cooper, who had been working on preliminary designs, proceeded to England with the Engineering Travelling Scholarship and arrangements were made with Stressed Concrete Design Ltd. for him to work in their office on final details of design. In the event Mr. Cooper made all final calculations and sent out a number of sketch drawings. All detailed construction drawings were made in Wellington and sent to Stressed Concrete Design Ltd. for check. They were found to be complete and adequate.

The number and size of the I-beams depend on practical considerations. If they were bigger, the handling would be more difficult. If they were much smaller they would be too slender to handle safely, though it is not easy to establish what are safe limits.

With the size of I-beams selected so as to keep longitudinal stresses within satisfactory limits, the amount of high-tensile wire is established. One of the attractions of prestressed beams is the small quantity of high-tensile steel required. It is, however, not so often mentioned that the quantity of ordinary reinforcing steel can be quite considerable. Photographs of the Walnut Lane beams show a close arrangement of stirrups following the outline of the web and bottom flange of the beam. For the Hutt Estuary Bridge there was no theoretical need for any reinforcing steel except in the end blocks. For the first few beams none was used, but later, enough reinforcing steel was used in the form of stirrups to take the theoretical shear for the end panels. Owing to the parabolic cable effect, $\frac{3}{8}$ in. stirrups at 12 in. were adequate and the total amount required was not great. For reasons of fixing rubber cores, $\frac{3}{8}$ in. stirrups at 18 in. were used for the remainder of the length of the beams. The criterion for shear in prestressed webs is, of course, the maximum principal tensile stress, but it is somewhat disconcerting to be told that the safe limit figure is 100 lb./sq. in., or whatever the figure may be, with no supporting evidence.

2.1.1. *Beams: Bending*

The I-beam section operates under three conditions (the centre section only is considered here):

A: Net concrete as cast, with no grout (the cables act as a simple external force).

B: Beam free (cables grouted as in test load).

C: Beam in bridge (extra deck between flanges).

Assuming a modular ratio of 7 the properties are:

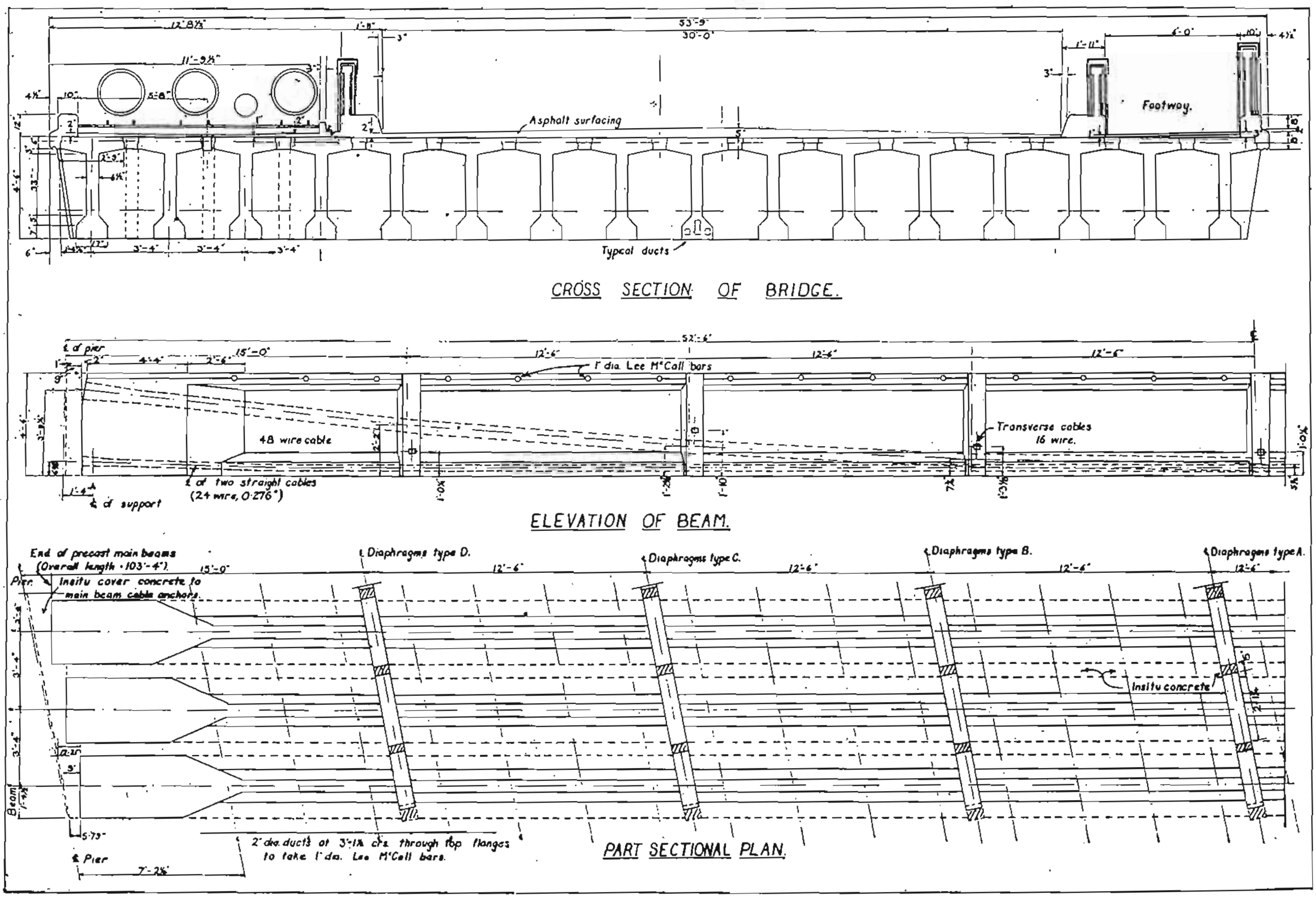


Fig. 4.—Superstructure.

	Moment inertia	yt	yb	zt	zb
A	207,768	21.9″	32.1″	9,470	6,460
B	258,678	24.8″	29.2″	10,440	8,880
C	278,973	23.5″	30.5″	11,860	9,150

With 96 wires of 0.06 sq. in. (0.276 in. diameter) stressed to 120,000 lb./sq. in. the prestressing force is 690,000 lb. (307 tons) at an eccentricity of 27.48 in., giving a moment of 19,000 k'' (k = 1,000 lb.).

When a beam is first stressed, the condition, assuming no loss of prestress, is (stresses in lb./sq. in.):

		Top fibre	Bottom fibre
Stressing force 690k		+ 1,130	+ 1,130
Stressing B.M. 19,000k''		— 2,020	+ 2,940
Dead load B.M. 12,000k''		+ 1,270	— 1,860
	Result	+ 380	+ 2,210

In this condition the beam is grouted and the properties B apply. One beam was test loaded in this state with 40 diaphragms each weighing 1,500 lb. or a total of 26.8 tons, so disposed as to cause a bending moment of 10,700k''. Again, assuming no losses, the stresses were:

		Top fibre	Bottom fibre
As stressed		+ 380	+ 2,210
Test B.M. 10,700k''		+ 1,020	— 1,210
	Result	+ 1,400	+ 1,000

On the assumption that 15 per cent. loss of prestress had occurred, and that grouting had not been fully effective, the final stress in the bottom flange would have been negligible or a slight tension, and minute cracks might have been discernible. Probably the condition was more like that set out above. The difference caused by various assumptions is, however, worthy of note.

When the beams are incorporated in the bridge they are infinitely more stable laterally and at that stage properties C apply. The figures below take account of the extra dead load due to roadway deck structure (not kerbs and handrails) and of H20—S16—44 live load with 22 per cent. impact divided among three beams or 10 ft. of width. A loss of prestress of 15 per cent. is assumed.

These figures demonstrate the modest calculated stresses in the concrete under live load, and even these stresses are not likely to be realized, owing to lateral distribution through the diaphragms. The top flange, in which maximum live load stresses occur, is also prestressed transversely. In the light of the concrete test results, the factor of safety of the beams as incorporated in the bridge is probably at least three. The figures also make it quite clear that the most severe test of the beams is for bottom flange compression at the time of stressing. See Fig. 5 for calculated concrete stresses.

	Top fibre	Bottom fibre
Stressing force 585k	+ 960	+ 960
Stressing B.M. 16,150k''	— 1,710	+ 2,480
Dead load as cast: 12,000k''	+ 1,270	— 1,860
Result: Properties A	+ 520	+ 1,580
Dead load deck, 3,200 k''	+ 270	— 350
Assumed permanent condition	+ 790	+ 1,230
Live Load: 7,700 k''	+ 650	— 840
Total with Live Load	+ 1,440	+ 390

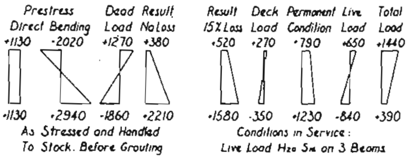


Fig. 5.—Concrete stresses in beams—lb./sq. in.

The steel wires are stressed to 120,000 lb./sq. in. as stressed. Assuming a 15% loss by the time the beams are placed, the stress is then 102,000 lb./sq. in. From this stage to full live load the concrete stress in the bottom flange increases in tension by 1,190 lb./sq. in. According to their distance from the neutral axis and with a modular ratio of 7, the increase in steel tension will be 7,800 lb./sq. in., making a total of 109,800 lb./sq. in. and confirming that in service the wires are never likely to be as highly stressed as at the time of stressing. See Fig. 6 for a typical test of the high-tensile wire.

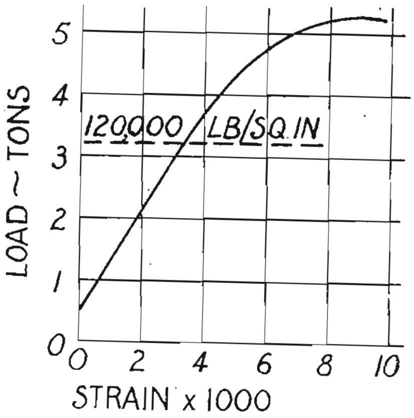


Fig. 6.—Typical test of 0.276 in. dia. high-tensile wire (Area = 0.06 sq. in.)

The calculated figures show that overload due to a 100% increase of live load in one lane will not cause undue stresses in the top flange, but that there would be a slight tension in the concrete of the bottom flange. The main interest regarding overload is the effect on the transverse diaphragms.

Space does not permit analysis of bending in the beams at sections other than mid-span.

2.1.2. *Beams: Shear*

Shear in the main beams may be considered as follows, for a section where the end block meets the web or 7 ft. from the end of the span:

Dead load: 44.16′ at 970 lb.		= 42.9 k
Live load $H20$ — $S16$ — 44	= 58.6 k	
Impact 22%	= 12.9 k	
Total:	71.5 k	
Per beam of three beams =		23.8 k
	Total:	66.7 k
Shear taken by parabolic cable, assuming 15% loss		= 30.6 k
Shear to be taken by concrete		= 36.1 k

By conventional methods, and for comparison only, mean vertical shear on the concrete web of 54 in. x $6\frac{1}{2}$ in. is 103 lb./sq. in. In terms of principal stresses, maximum horizontal shear at the neutral axis:

$$v = V\ A\ y / I\ b = 130 \text{ lb./sq. in.}$$

Direct stress on neutral axis with 15% losses $c = 960$ lb./sq. in.

$\therefore$ Principal tension is

$\sqrt{(V^2 + C^2/4)} - C/2 = 16$ lb./sq. in.

If overload shear is taken at twice the live load and on only two beams:

Dead load	=	42.9 k
Overload	=	71.5 k
Total:		114.4 k
Less parabolic cable		30.6 k
Shear on concrete		83.8 k

Then horizontal shear at neutral axis $v = 302$ lb./sq. in.

Direct stress as before $c = 960$ lb./sq. in.

$\therefore$ Principal tension is 86 lb./sq. in.

The safe limit for principal tension is generally assumed to be 100 lb./sq. in., and it is most unlikely that in service the principal tension will ever reach 50 lb./sq. in.

2.1.3. *Diaphragms*

The diaphragms serve to distribute loads laterally, and, even though individual beams may be strong enough for all loads, their deflection must be restrained and made to conform to a suitable pattern for the bridge as a whole. The diaphragms are parallel with the piers, and thus on a skew of 11° relative to the beams. A rigorous analysis would be extremely involved. More elaborate calculations have been attempted, but the following simple figures are submitted:

If two $H20 - S16 - 44$ vehicles are situated in mid-span their total weight is 144 k or 65 tons. The bending moment induced in the beams is approximately 37,800 k, not including impact. If this load is spread over 6 beams the moment per beam will be 6,300 k'', and by analogy with the test beam the deflection would be, say, 0.8 in. If the load were spread uniformly over all of the 16 beams in the span, the deflection would be 0.3 in. Then, if it be assumed that the deflection in the centre of the roadway is 0.5 in. (instead of 0.8 in.) and the reflection of the outer beams 0.2 in. (instead of the mean of 0.3 in.), the relative deflection of the middle diaphragm would be 0.3 in. in a length of, say, 50 ft. The relative deflection must be less than 0.5 in., and if the diaphragms were stiff enough it would be zero. Assuming, however, 0.3 in. in 50 ft. on a circular curve and with no compression of the deck, the extension of the bottom fibres would be such as to cause a tension in the concrete of 1,650 lb./sq. in.

Each diaphragm is prestressed by means of 16 wires at 140,000 lb./sq. in. Assuming 15% losses, the prestressing force is 114 k, and if applied uniformly to the section of 45 in. x 9 in. the compression in the concrete would be 280 lb./sq. in., and on a triangular distribution it would be 560 lb./sq. in. at the bottom. Such a basis of calculation would indicate tension or cracking of the bottom flange under the live load described.

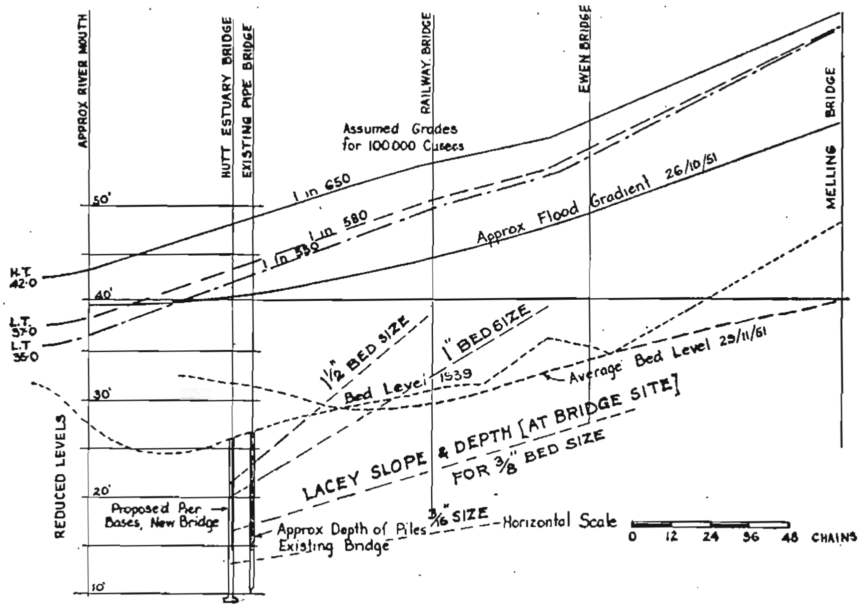


Fig. 7.—Flood gradients.

As to steel stresses, the curvature of the diaphragm discussed above would cause an increase of stress of something less than 7 $\times$ 1,650, or, say, 10,000 lb./sq. in., which would not overstress the wires.

The position is complicated by the fact that initial prestressing of the diaphragms must cause the deck to camber transversely, and subsequently the transverse stressing of the deck must reduce this camber. Unfortunately, no measurements of deflection were taken during the stressing of diaphragms or deck.

Working back from the stiffness of the diaphragms, the moment of inertia of each, including 6 ft. of deck, is 190,000 in.^{4}, and the moment required to induce the curvature assumed is therefore 5,800 k''. Every diaphragm in the span will be affected, but if the middle diaphragm takes a load of 30 k at each of two points 10 ft. apart, and distributes these to virtual reaction points 40 ft. apart, the bending moment would be 5,400 k'', which corresponds roughly with the curvature and indicates that the crude assumptions made are somewhere near the truth. The problem is, of course, one of relative deflections.

2.1.4. *Deck Slab*

The most effective position for the transverse prestressed tendons would be just below the slab, and initial proposals showed this arrangement. However, no satisfactory way of protecting the tendons against corrosion could be devised, and it was decided for this reason to incorporate them in the slab.

The design allowed for 8 wire cables at 3 ft. centres stressed to 140,000 lb./sq. in. to give a force per cable of 67 k over an area of 36 in. x 6 in. The transverse prestress in the concrete is initially 310 lb./sq. in., reducing after 15% losses to 263 lb./sq. in.

It is difficult to conceive of the slab failing in any other manner than by local diagonal tension under a concentrated wheel load. The axle load of $H20 - S16 - 44$ loading is 24,000 lb., giving a wheel load of 12,000 lb., and this may be spread over 20 in. of width. Dual wheels with a tyre pressure of 100 lb./sq. in. would require 120 sq. in. bearing on wheel prints about 6 in. wide and 10 in. long each. The area covered by the dual print would thus be about 16 in. x 10 in., and, allowing 3 in. all round for the effect of the asphalt surface, the area which might be punched out of the slab is 22 in. x 16 in. with a periphery of 76 in. The unit shear for the 6 in. of slab with a 12,000 lb. load on this periphery would be 26 lb./sq. in., which is a very low figure.

Viewed from another angle—with a prestress of 263 lb./sq. in., the shear necessary to produce a maximum principal tensile stress of 100 lb./sq. in. is 190 lb./sq. in. In order to produce vertical shear of this intensity, a 12,000 wheel load would have to be concentrated on an area of about 2½ in. square, which requires a unit bearing of 2,000 lb./sq. in., and is virtually a point load.

The foregoing ignores bending of the slab and peripheral distribution of shear in the depth of the slab. It is intended only to prove that the slab is safe.

2.1.5. *Frequency*

At the suggestion of the chief designing engineer of the Ministry of Works, the frequency of vibration of the spans was calculated as set out in R. G. Norman's paper (*N.Z. Eng.*, 5 (3), 239-43 (1950)).

Dead load per beam = 971 lb./ft. run.

Calculated deflection $d = 5/384 \cdot WL^3/EI = 1.76$ in.

For $E = 4.6 \times 10^6$ and $I = 278{,}973$ in.^{4}.

Then frequency $= 3.53 / \sqrt{d} = 2.65$ cycles per second.

Very little has been published on the subject of vibration, but there is evidence that simply-supported spans of 80 ft. to 150 ft. may have an undesirable frequency. This range of spans is that in which prestressed construction may be considered advantageous in other respects.

2.2. Piers

On looking back, it is open to question whether the piers should not have been carried to foundations below the clay layer, accepting whatever risks might be involved in penetration of the artesian seal. The design problem was to sink them deep enough to be safe against scour, yet to leave enough granular material above the clay to distribute the foundation load and reduce settlement.

The material above the 16 ft. layer of marine clay consisted of shingle, sand, shells and seams of conglomerate or clay. These will be mentioned under Part 3, "Construction", as they cause conclusions from the test bores to be modified somewhat.

When the piers were designed, however, it was considered that they might be founded at R.L.10 based on preliminary calculations of scour, and that as regards settlement the determining factor was the 16 ft. layer of clay. The report from the Soil Bureau gave the shear strength as 18.5 lb./sq. in. (or 1.2 ton/sq. ft.). The bearing pressure on the pier bases was taken as 2.5 ton/sq. ft. at a level 10 ft. above the clay.

Then, as calculated by the Soil Bureau:

	ton/ sq. ft.
Increase in pressure at top of clay layer	= 1.54
Increase in pressure at bottom of clay layer	= 0.59
Mean increase of pressure in clay	= 1.06
Overburden pressure at top of clay	= 1.40
Overburden pressure at bottom of clay	= 1.76
Mean overburden pressure in clay	= 1.58
Mean final pressure in clay	= 2.64
Initial voil ratio e	= 1.064
Final void ratio	= 1.035
Difference δe	= 0.029

Therefore settlement $= \delta e / (1 + e)$
$= 0.029 / 2.064$ or 2.7 in. in 16 ft.

The consolidation test curve is shown in Fig. 2. The clay was found to be over-consolidated to a degree which would be consistent with an additional overburden of 100 ft. at some stage of its history.

The predicted settlement was regarded as acceptable, and attention was again directed to the problem of scour. Figure 7 shows the "normal" bed level of 1939, and the measured bed level after a 32,000 cusec flood in October, 1951. The diagram also shows a probable flood grade at high water and low water for the assumed maximum discharge of 100,000 cusecs. The high flood level at the Ewen bridge is in accord with the calculations of the Hutt River Board, and the bridge is very near the outlet of the river to the sea. Therefore the flood slope is fairly well established at 1 in 650 (to 1 in 550). The tendency is for the bed to degrade, so 1 in 650 is taken as a general figure for slope, though there will be a local increase in slope at the bridge site owing to a restriction in width and the effect of the piers. The nature of the bottom at present gives no evidence as to scour, since the deeper layers above the clay are obviously of marine origin and have never been scoured. The piles of the old pipe bridge 150 yd. upstream are not deep (as shown), and have stood in a recorded flood of 70,000 cusecs. This, again, means little in relation to the new bridge.

	Level	Slope
High tide	47.6	1 in 660
Low tide	43.0	1 in 580

Observations of the surface material of the bed shows a gradual increase in size from the Estuary Bridge to Maoribank, some 13 miles upstream, being noted as under:

(a) Estuary Bridge: The whole exposed area of the west bank is paved with shingle of 1 in. to 4 in. diameter, and the ruling size (at 60% of the total) is taken as 1½ in.

(b) Railway Bridge: An exposed shingle bar in the middle of the river shows material larger than quoted for the Estuary Bridge site.

(c) Melling Bridge: At the edge of the low flow channel, stones from 1 in. to 6 in.—say, 60% at 3 in.

(d) Maoribank: In the rapids are many stones of 12 in.—say 60% at 6 in.

The river brings down quantities of shingle which gets worn to a smaller size in its passage to the sea. Shingle is removed in considerable quantities for commercial use near the Melling Bridge.

So much for ascertainable facts. Many papers have been written on "stable channels in alluvium", and this subject has a bearing on scour, but not nearly enough is known about scour. Flood peaks in the Hutt River are of short duration, and bed material is being tumbled down a gradually decreasing slope. It is debatable whether "stable channel" formulae apply. Furthermore, the many papers, by Lacey and others, on "stable channels" appear to be concerned with irrigation channels where slopes are much flatter and bed material much finer than at this bridge site.

However, the problem must be attacked somehow, and it will be treated as if the bed material were all non-coherent and of uniform size. Two reservations may be made: the first, that at piers 1 and 2 clay was found in the test bores at a higher lever than the main clay layer; the second, that in a bed material of graded material the finer fraction might soon be carried away, but it would leave a virtual armour coating of coarser material.

On the simplest basis—*i.e.*, Manning's formula,

$$V = 1.486 / n \cdot R^{2/3} \cdot S^{1/2}$$

For $n = 0.03$, and for a width B of 512 ft., the following are derived:

V av.	V max.	d av.	d max.
12.3	15.4	16.2	24.3
12.8	16.0	15.6	23.4

The maximum depth above would indicate general scour in mid-river to about R.L.20 with a bottom velocity of, say, 5 ft./sec. Hunter Rowse gives a terminal velocity of 2.5 ft./sec. for 10 mm. or ⅜ in. particles, so that if 1½ in. stones form the bed they would be stable. Such stones are not present in quantity at the scour level calculated, but it seems fair to suppose that they will be brought down the river or separated out from the final material when it is removed.

Another approach is from Lacey's formulae:

Particle size in mm. = m.

Friction coefficient $f = 1.76 \sqrt{m}$.

Mean velocity $Vm = 0.794 \times Q^{1/6} \times f^{1/3}$.

Unrestricted width for $Q = 100{,}000$ cusecs:

$B = 2.67 \sqrt{100{,}000} = 845$ ft.

Actual width B (restricted) = 512 ft.

Slope $S = 0.000547 \times f^{5/3} / Q^{1/6}$.

Average depth d (unrestricted) $= Q / B\,Vm$.

d (restricted) / d (unrestricted) $= (B/B \text{ res.})^{2/3}$

$= 1.395$ for $B = 512$ ft. (res.).

Then for a range of particle size:

m (mm.)	in.	f	Vm	d unres.	d res.	d max.	S:1 in.:
1	—	1.76	6.53	18.1	25.3	38.0	4,850
5	—	3.94	8.54	13.8	19.3	29.0	1,270
10	⅜	5.57	9.58	12.3	17.2	25.8	710
25	1	8.87	11.20	10.6	14.8	22.2	328
38	1½	10.87	11.97	9.9	13.8	20.7	233

The maximum depth by the Manning formula takes no account of particle size except as it may be involved in the roughness factor n. This formula gives a maximum depth of 23.4 ft. for a slope of 1:580. The Lacey formula interpolated for the same slope of 1:580 (from the table above) gives a maximum depth of 25 ft. which corresponds with a particle size of just over $\frac{3}{8}$ in. To this extent the formulae are in reasonable agreement. In both cases maximum depth has been taken as 1.5 times mean, which assumes a parabolic cross-section. It is, however, a rather extreme assumption, and, if the ratio be taken as 1.3, then maximum depth by the Lacey formula would be, not 25 ft., but 21 ft. 6 in. From a flood level at low tide of R.L. 43.0, the bed level at mid-river would then be R.L. 21.5. The piers were designed to be based at R.L. 10 or 10.5 ft. below this calculated general scour level. Some margin must be allowed for local scour, but, owing to their length, the piers would be stable even if the upstream ends were scoured to a level only a few inches above the base.

Returning to the Lacey formula and Fig. 7, arrows are drawn to show particle size and slopes for the depths tabulated. The probable slope corresponds with a particle size of just over $\frac{3}{8}$ in. as already stated. It is difficult to imagine the slope ever being substantially greater than 1 in 580 (except for local effects), but if sufficient shingle is taken from the river it might become flatter. Even so, with one end of the slope curve tied to sea level, the waterway would then be deeper and the velocity less. There does not seem to be any process by which the river could remove the material from $\frac{3}{8}$ in. upwards to a dangerous level.

It is of some interest to consider the theoretical effect of a flood of 80,000 cusecs, which is slightly above the record of 1939. For the 10 mm. size, the calculated maximum depth would be 22.8 ft. instead of 25.8 ft. (both on parabolic section). If taken on the 1.3 ratio, the depth would be 19.7 ft., or to R.L. 23.3. For pier bases at R.L. 10 the margin would be 13.3 ft. from general mid-river bed level to pier base.

This section of the paper has described the calculations for scour which were made before the work was begun. Further comment is made in describing the construction of the piers. It is suggested, however, that a general approach to scour should be:

(a) To be safe, with a sure margin.

(b) To adopt a type of foundation in which extra depth, if required, can be added at small cost.

(c) To consider the cost of extra depth as a percentage of the total cost of the project. It most cases it will be small, and the designer will be encouraged to sink the foundations deeper.

Apart from bearing pressure and scour, the pier design presented few problems. The piers are designed to resist horizontal forces of 0.1 g due to earthquakes; also of forces due to temperature. The central span is fixed to the piers at each end, so that any movement due to temperature must be accommodated largely by the piers in flexure. To some extent it will be absorbed by stresses in the span—*i.e.*, when the span expands it will tend to become prestressed to a greater extent. Other spans than the central span are fixed at the shore end in each case and have roller bearings at the other end which, owing to the longitudinal camber, is the uphill end. Heavy and close reinforcement is incorporated in the pier cap to ensure distribution of local high stresses under the beam ends.

3. CONSTRUCTION

3.1. Making Beams

The beams, 80 in number, had to be concreted, stressed, moved to stock and subsequently placed on the bridge. The beams could have been concreted in one length, but the precasting of diaphragms offered the advantages of locating the rubber cores and of dividing the concrete into compartments so that long "live" faces of concrete would be avoided. Shrinkage would also take place on the 12 ft. sections between diaphragms and be more likely to form gaps against the diaphragms than to crack the beam section.

Diaphragms were thus precast in advance of the beams, and set up on the beam beds. Beam shutters were fixed between diaphragms.

Though the beams were all identical in their main features some were exterior beams, some had kerb steel projecting, and the expansion rollers had to be provided for at one end or the other. Thus there were in all twelve slightly different types of beam to be cast. The sequence in which they should be cast depended on which spans were to be placed first. It was possible that a beam might be rejected or damaged in handling. The layout therefore had to be such as to allow beams to be taken from stock not necessarily in the order in which they had been cast. The initial plan provided for beams being taken endwise from the casting beds so as to allow a degree of selection. The plan adopted was to have two sets of beds, 10 beds in one and 6 beds in the other. From these the beams were skidded sideways to stock along double-rail "runways". From the runways the beams were slid on to bogies on a 6 ft. gauge track leading to the access bridge. It was thus possible, if a certain beam was wanted, to "backshunt" from one set of runways to the other, though in practice it never had to be done. Figure 8 shows the layout of the casting beds.

The beam beds were of concrete 6 in. thick set to the exact outline of the beam base. The diaphragms were set up on the beds and the beam moulds fixed in between. The first few beams were cast directly on the concrete base, but before long (as is mentioned elsewhere) a false timber floor was used. A beam mould was made of steel plate $\frac{1}{8}$ in. thick, suitably stiffened, and from this mould 66 beams were made. At a later date a timber mould was also made partly in order to increase output, but mainly in order to enable a more satisfactory schedule of labour operations to be organized. Great pains were taken in the design of the timber mould to avoid certain defects in the steel mould, but it was not as satisfactory as the steel mould, and only 16 beams were cast in it.

Difficulty was experienced in sealing the steel mould against the diaphragms, and grout leakages occurred. The vibration of the moulds contributed to the difficulty, and careful inspection was necessary at all stages to avoid grout

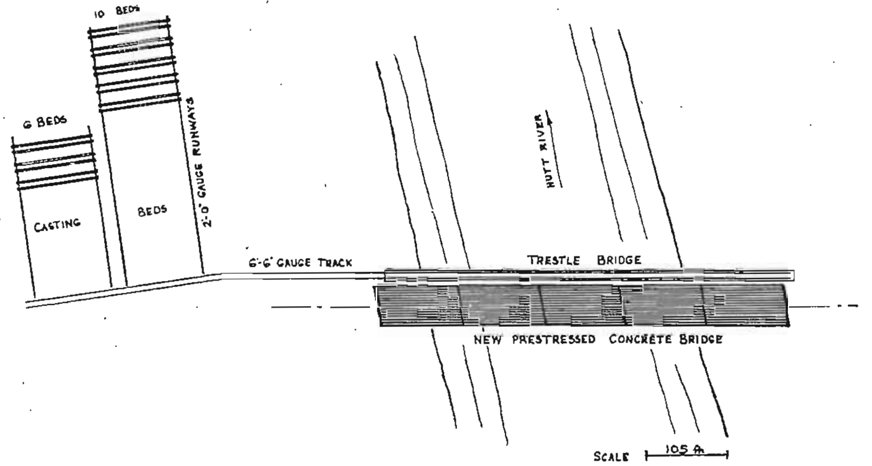


Fig. 8.—General plan of site.

leakage. The most satisfactory seal was made by caulking the vertical joints with shreds of rope and wax. In a few of the beams, but notably in the first one, cracks were seen at the junction of the web and top flange. These were at first thought to be shrinkage cracks but they were really caused by the web concrete settling slightly while the top flange was being poured. The cracks did not penetrate and did not occur in many beams.

The holes in the top flanges for the transverse prestress were formed for the steel mould by sleeves of rubber radiator hose over steel pipe spreaders. The holes so formed were irregular in size and usually too big. When the timber mould was made, great pains were taken to locate the holes accurately and they were formed by means of inflated "Ductube". Later, when the beams were placed on the spans, it was found that errors had still crept in, and that some beams hogged nearly 1 in. more than others. The oversize holes formed by the radiator hose were then very welcome.

Devices for holding the rubber cores for the main cables were as illustrated in Fig. 9. They proved to be quite effective, but expensive. No special difficulty was experienced in withdrawing the rubber cores. In a few cases the cleaning-out of the ducts was troublesome, but in most cases the cables were pulled through easily. Stressing proved to be very simple. No wires were broken, and in very few cases did the wedges slip. When they did, of course, the position was rectified.

The main cables were grouted with a "Colcrete" mixer using a mix of 10 shovels of dry sand, 1 cwt. cement, and 6 gal. of water.

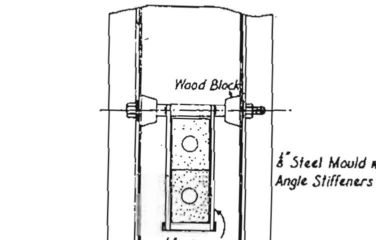
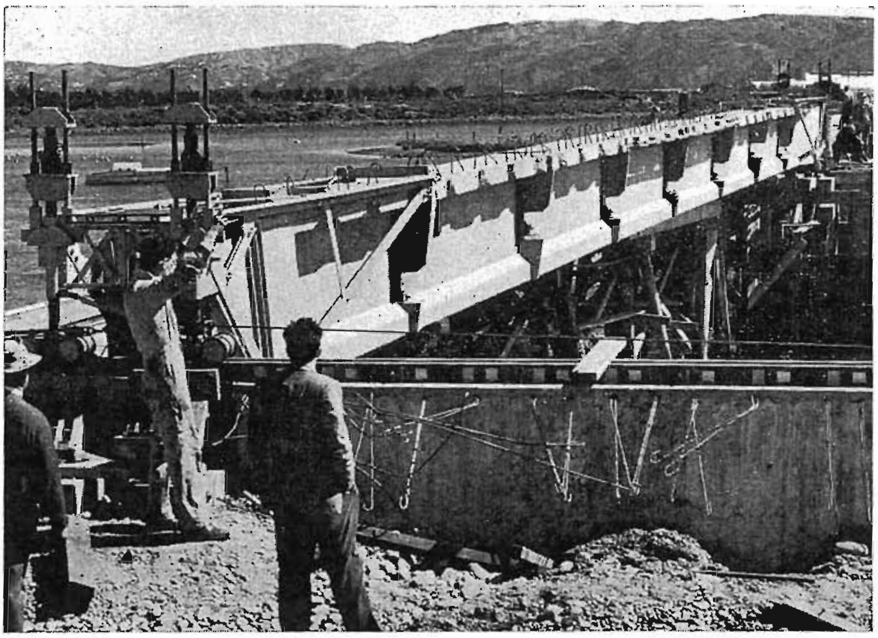


Fig. 9.—Fixing to rubber cores.


First beam being rolled on to completed piers.

Several simple tests were made of grout settlement in glass jars which showed a separation of $\frac{3}{8}$ in. in 6 in. depth.

The writer is confident that grout was forced through the cable ducts so as to fill all of them effectively. At the ends of the parabolic ducts, the grout usually settled back a few inches, and this was made good. The disquieting thought remains as to what can be done afterwards if the grouting of prestressed wires is not perfect.

3.2. Handling Beams

The first few beams were lifted on a frame made up from the tyres and undercarriage of an aircraft. It was not successful, and five beams were then lifted off their beds by a "cherry picker" and placed on Athey wagons for transport to stock. The beams were towed by a tractor over rough ground and, in spite of warnings as to the need for keeping the beams vertical, one of the Athey wagons lurched into a depression in the ground. The beam which was being towed virtually exploded under the release of prestress, and in so doing "touched off" another beam which was already in stock. This also broke into fragments.

After this incident it was insisted that the beams (which weighed 40 tons apiece) should be handled on steel rails set on sleepers, so that the whole process would be under control. Double-rail runways on half sleepers were laid a few feet from the ends of the beams running transverse to the beds. The beams were jacked up and placed on hardwood palettes heavily greased on the under side. This process would have been easier if provision had been made beforehand for permanent jacking bases or even for jacking pits under the beam ends. The beams were then pulled sideways to stock. Static friction was surprisingly high; the weight on one beam end was 20 tons, and it is estimated that 5 to 7 tons pull was sometimes required to start movement.

A few beams were pulled sideways by hand winches, but, as might be expected, a change was soon made to mechanical power, and steam winches were selected as giving the best control. Even so, with a long length of steel wire rope in tension, the beams started moving with a jerk which was quite alarming to the operators (and perhaps not only to the operators).

The beams were surprisingly flexible laterally and could be induced to oscillate with a range of about $\frac{1}{2}$ in. by the pressure of one hand. If they started moving sideways with a jerk, the oscillation appeared to have a range of 3 to 4 in. It would have been comforting to know to what extent the beams could oscillate without danger. The writer could not devise any method of determining the safe limits without a test to destruction. The bottom flange would not readily buckle, since it contained the cables, but grout leakages had caused weak points here and there at the contact between the bottom flanges and the diaphragms, and these could be overstressed by lateral movement. The top flange had no longitudinal reinforcement whatever, and no connection, other than by pressure, between the top flanges and the diaphragms. Perhaps the beam was amply safe. Various devices by way of temporary stiffening frames were discussed, but nothing was done and nothing untoward occurred. Several different types of grease were used with a view to reducing friction. The unit pressure of the hardwood palettes on the rails averaged 300 lb./sq. in. on side grain.

3.3. Placing Beams

The scheme envisaged before the contract was let was to drag the beams on steel rails to a loading dock and then to float them into position on rafts of oil drums or similar containers. The tide would have assisted, and air could have been pumped into the drums to give vertical movement. However, there was not enough water to float the end spans without dredging, and these might have required other methods. The beams were safe for a lift 10 ft. from each end.

However, the contractors elected to construct an access bridge which would be of great assistance in working on the piers, as well as in transporting beams. It is an open question whether it would not have been preferable to float the beams, but the position of the access bridge ruled out this possibility. After the access bridge was built, various methods of placing the beams were investigated, including cableways, but it seemed fairly obvious that the beams should be run out longitudinally on to the access bridge and then traversed into place on the piers. The 11° skew of the piers had to be allowed for. The pier top was only 3 ft. wide, and it was at first considered impossible for the beams to be traversed on the pier top alone. At one stage it was proposed to fix temporary brackets to the piers from which the beams could be jacked and adjusted, and holes were left in the first pier for this to be done. However, jacking from below would have placed the operator in a possibly dangerous position and made it very awkward to place the roller bearings, and so it was decided that a way must be found to satisfy two conditions — (a) the beam carriages must be carried on the pier tops between beam ends; (b) the jacks must be operated by a man standing on the top of the beam.

Traversing cradles were eventually contrived which met these requirements though clearances from the sandwich plates of the cable anchors were fractions of an inch (see Fig. 10).

The beams were therefore skidded from the runways on to bogies on the 6 ft. track, and pulled along the track until they were opposite the span in which they were to be placed. Owing to the scheme being developed in stages, the various levels of runways, access bridge and piers required the wheels for the bogies to be of 6 in. external diameter. The wheels were of sturdy construction, with a load of only 5 tons each, and with tapered roller bearings. Speed was negligible, and the total distance to be travelled was small, yet the wheels were gradually squeezed as is pastry under the housewife's rolling pin. The diameter enlarged and the bearings became slack. Slightly stiffer wheels were made of a better grade of steel, and no further trouble was experienced.

When the beams reached the position from which they were to be traversed (at 11° skew), rails were set in line with the pier tops and the traversing cradles were fitted to the beams. The cradles were then jacked up to release the 6 ft. bogies, and the beams were traversed along the pier tops into position. Some care was given to keeping the beams parallel with the bridge axis, since the effect of the skew would be that the wheel flanges would bind if the beams got off line. However, the beam carriage wheels were on roller bearings, and the traversing proceeded very smoothly throughout. On occasion more than one beam was placed in a day, but a general rate of one beam per day was a good performance.

The design of the cradles allowed the roller bearings and end seatings to be fixed quite readily. The cradles, in fact, gave good control for lifting and traversing, but, however carefully the rails were placed, the beams could not be set accurately to their final position longitudinally. It was necessary to make another device for adjusting longitudinally to the last $\frac{1}{8}$ in. This longitudinal movement had seldom to be more than $\frac{3}{4}$ in., but accurate control was necessary to get the holes in the top flanges to register (see Fig. 10).

The placing of the beams on to the spans proceeded quite smoothly and according to plan. There was, however, the unpleasant possibility that, if an accident occurred, such as the failure of a beam cradle, the beam in transit might explode, and set off a chain reaction on the beams already placed in the span concerned. To minimize this risk, the top beam flanges were bolted together in succession as they were placed. A strong southerly gale came up the night after the first beam was placed on the span, and it no doubt made the beam quiver.

3.4. Transverse Prestress

Each beam had seven diaphragms in its length, with a $3\frac{1}{2}$ in. x $2\frac{1}{2}$ in. hole in each diaphragm for a 16-wire cable. (The holes could have been $2\frac{1}{2}$ in. x $2\frac{1}{2}$ in. except that it was convenient to

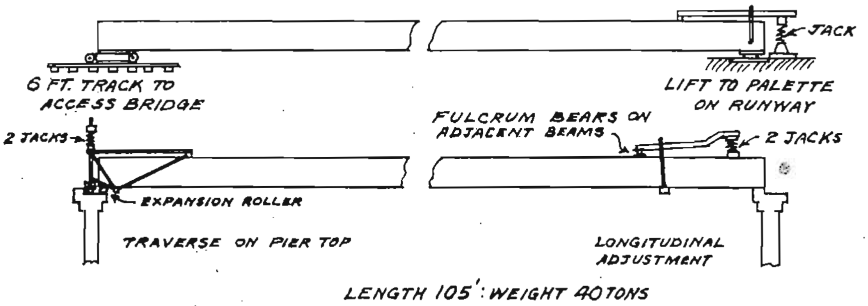


Fig. 10.—Beam-handling.

use the rubber core section from the main beam cables.) When the beams of one span had been assembled it was necessary to place concrete between the precast diaphragms so as to form seven continuous diaphragms across each span. The hole for the prestressing cables had, of course, to be continued through the *in situ* concrete. The first attempt to form this hole was made using short lengths of rubber core vulcanized on to steel rods, but the holes which had to be engaged were not truly in line, and the rubber core proved very hard to extract. Attempts were made to have a stiff rectangular inflatable tube made in short lengths, but this could not be arranged. Finally, the job was done in the simplest possible way—that is, by concreting above and below the line of the cables and passing the cables through the gap. They were then stressed and wrapped with adhesive tape to keep concrete out while the gap was being concreted. Grouting then proceeded in the usual way.

The prestressing of the top flange was designed for 8-wire Magnel cables and 2 in. holes were formed accordingly at 3 ft. centres. As the work in general progressed it was considered that placing the many 8-wire cables would be slow, and that twisting would probably occur in the circular holes. This apprehension may have been unjustified, but, nevertheless, 1 in. diameter Lee McCall bars were ordered in lieu of the 8-wire cables. These were used in two lengths of approximately 27 ft. for each tendon with a coupler in the middle and the usual "high efficiency" nut at each end. Assembly and stressing proved to be very simple, the only delay being caused by the variations in the overall width of the bridge. To accommodate this, the anchor plates had to be packed out with mortar, or, alternatively, cut into the top flanges. With Magnel wires, of course, this adjustment would have been automatic.

There was, however, an unhappy period when it was first seen that the top flange holes did not register. Discrepancies were due to accumulated errors in making and placing the beams, and due to variation in the "hogging". At first it seemed as if the holes would have to be bored out from the ends, which would have been a most tedious process. Investigations were also made as to the practicability of sawing out the holes where necessary to form grooves—*i.e.*, where the beams were low. However, once a quick, safe and controllable method of adjusting the beams longitudinally had been devised, it was seen that the situation was not as serious as had appeared, and of 30 rods in each span it was usually possible to thread, say, 20 without resorting to any cutting away. The rods which could be threaded were therefore placed and stressed, omitting the beam flange concrete on the lines of the rods which could not be placed. Where necessary, the concrete above the holes was then cut away with a pneumatic pick to enable the rods to be inserted. It was regrettable that some prestressed concrete had thus to be removed, and advice was sought as to whether it could be made good with a concrete which would expand enough to reimpose the prestress. Advice from M. A. Craven, engineer to the N.Z. Portland Cement Association, was, however, that no process was available which would cause the concrete so placed to expand to a predictable extent. Ductube was used for forming most of the holes in the concrete between deck flanges. It might be argued that flexible wires could be threaded through holes which are out of alignment much more easily than the rigid 1 in. Lee McCall bars. This is so, but any appreciable deviation from a straight line would result in most undesirable lateral components of stress, and the vertical components could easily be of sufficient magnitude to spall off pieces of the deck after the bridge was in service.

The formwork for the concrete covering the anchors for the diaphragm cables and the Lee McCall nuts had more or less to be hung in space. Various schemes were drawn up and for the type adopted it was found that "Ramset" studs proved very useful.

3.5. Pier No. 4

This pier was the first to be constructed. Sheet piles of Frodingham No. 2 Section 50 ft. long were driven to form a rectangular box approximately 62 ft. by 20 ft. in plan. The piles were driven with a land frame from a staging, using a 9B3 McKiernan Terry hammer. Driving was slow throughout. The piles were, as is usual, pitched in panels and driven in stages, but the overall rate was not quite one pair of piles per day. Closure of the corner piles was made satisfactorily. Excavation and timbering proceeded without difficulty. For part of the time a gravel pump on the ejector principle was used. This worked quite well, and the cofferdam proved to be so tight that water had to be pumped in to supply the gravel pump. Seven frames of 12 in. x 12 in. timbers were used, the bottom pair being placed close together so as to leave a clear space for work on the pier base. The struts for the four lower frames were made of reinforced concrete so that they could be built into the pier.

Levels taken on the sheet piles as excavation proceeded showed an upward movement of $2\frac{1}{2}$ in. Prior calculations had been made of artesian pressure, and it seemed as if the factor of safety against a "blow" of the bottom were adequate. The uplift was regarded as puzzling, but not serious, and as soon as the first pour of concrete was placed in the bottom the uplift stopped. By the time the concrete for the pier was completed the pier proper had settled $1\frac{1}{4}$ in. and the cofferdam had settled $\frac{3}{4}$ in. The timber frames were replaced progressively by backfill as the pier shaft concrete was placed, and by the time the backfilling was completed the pier had settled an average of 2 in. in all while the cofferdam had settled a further 1 in., bringing it to within $\frac{3}{4}$ in. of its original level. The seats for the beams on the pier top were placed $2\frac{1}{2}$ in. high to allow for future settlement as calculated by the Soil Bureau.

After the sheet piles had been driven for the next pier (No. 3) attempts were made to extract the sheet piles of Pier No. 4 using a B.S.P. No. 80 extractor. Nearly a week was spent in extracting the first pile, but, since this pile engaged a clutch on each side, it was expected that progress would be slow. The pile showed quite clearly what part of its length had penetrated the marine clay, and the film of clay left in the clutches was very tenacious. A hand winch was rigged so as to apply a pull of 15 tons to the extractor, and this pull improved matters. The makers state that a pull of 2 or 3 tons is adequate, but with a pull of only 2 or 3 tons the vibration of the extractor parts seemed excessive. Extraction was proceeded with for several weeks and 13 single piles were extracted. The best performance was 3 per day, at which rate it would have paid to continue extraction. The force applied to the piles was such that pieces of the pile were pulled off by the extractor jaws on several occasions. Jets were tried, but they had no effect in the clay. Proposals were investigated for using floating plant with adequate headroom, but suitable pontoons could not be located. Pile extraction was therefore suspended while a B.S.P. No. 120 extractor was ordered. However, though the makers despatched the extractor promptly and co-operated in every possible way, by the time it arrived progress was such that pile extraction would have interfered with beam placing. The beam placing could have been deferred, but it was important to establish beam-placing methods, and too much effort on pile extraction would have detracted from progress on subsequent piers. The sheet piles were therefore cut off at bed level by a diver.

3.6. Pier No. 3

Sheet piles were driven as for Pier No. 4 and excavation completed. Levels were taken on the cofferdam every few days, and by the time the excavation was completed an upward movement of 1 in. had been recorded. This was less than for the previous cofferdam, but no explanation could be found for the sheet piles themselves rising as distinct from the bottom being forced up relative to the sheet piles. It was conjectured that the relief of weight on the general area had allowed the riverbed to be forced up by artesian pressure. Unfortunately, owing to bad weather, more than a week elapsed before any concrete could be placed. Waves 4 ft. high were noted, which may have caused working of the sheet piles. After the storm, work was resumed, and the screeds were being placed when it was noticed that the sheet piles had risen to such an extent as to tilt the walkway round them to an angle of 45°. E. D. Kalaugher, who was then in charge for the contractors, very properly and promptly ordered the pumps to be started so as to fill the cofferdam with water. The uplift was measured as just over 15 in. There could be no question of subsequent dewatering, and instructions were given to place 8 ft. of tremie concrete in the bottom. This was calculated to balance the artesian head, but, in fact, 11 ft. of concrete was placed, partly to level up the base and partly to accelerate settlement back to the original level. Figure 11 shows the position of the cofferdam relative to the artesian strata. It seems strange that the bed material forced up should have carried the sheet piles with it as a unit, and even more remarkable that the movement should be so local that the timber staging piles only 3 ft. away should remain at their original level. The clay appears to have failed in shear on vertical planes directly under the line of the sheet piling. It seemed reasonable to suppose that, since the uplift must have been caused by artesian water filtering through permeable sand and gravel, there would be a "blister" of water beneath the marine clay under the cofferdam. The pressure of such a blister would indicate rapid or unpredictable settlement as further weight was added to the pier.

In an effort to reduce the blister and to cause rapid settlement, 2 in. bores were made through the tremie concrete at each corner of the cofferdam. One of these bores went to R.L. 4 before striking clay, thus indicating that the artesian seal was not as thick as shown by the test bores. Pipes were set in the 2 in. bores, and capped, so that the release of water might be controlled. The pipes discharged water under pressure as was expected, but had no effect on settlement, even though at that stage there was a substantial calculated excess of weight over artesian pressure. Pier construction and backfill were proceeded with, but the construction of the pier cap was deferred until some evidence could be found which would indicate future settlement.

For purposes of calculation, the area of marine clay in shear was taken as the periphery of a rectangle 62 ft. by 20 ft. on an assumed thickness of 11 ft. below the sheet piles. This is an area of 1,800 sq. ft., so on the Soil Bureau figure of 1.2 tons/sq. ft. the out-of-balance pressure required to cause failure would be 2,160 tons. The upward pressure on the area of the cofferdam when the sudden uplift occurred was, for an artesian head of R.L. 52, some 2,370 tons. The weight of the plug under the cofferdam was calculated as 1,800 tons, so the net uplift on the plug was 570 tons, which would give a factor of safety against failure of the plug of 3.75. This figure was considered when the cofferdam was designed, but the "blow" had included the cofferdam itself, and, with the weight of sheet piles and timber added, the total weight of plug and cofferdam was 1,950 tons. The net calculated uplift was therefore 420 tons, giving an assumed factor of safety of 5, subject to the reservation made by the Soil Bureau as to the effect of remoulding on the clay.

It is interesting to conjecture what would have happened had the cofferdam not been filled with water. The weight of 1,950 tons would presumably have been raised until the artesian pressure was balanced at approximately 56 ft. head. This would have placed the underside of the plug at R.L. 4, or 12 ft. above its original level. Had movement of this magnitude been allowed to take place, the clay could have been so seriously disrupted that artesian water might have forced a passage round the periphery of the cofferdam. The resulting loss of water could well have had serious consequences for the whole of the Hutt Valley and affected 70,000 people.

Experiments have been made by the Wellington City Council by measuring the loss of head induced by heavy draw-

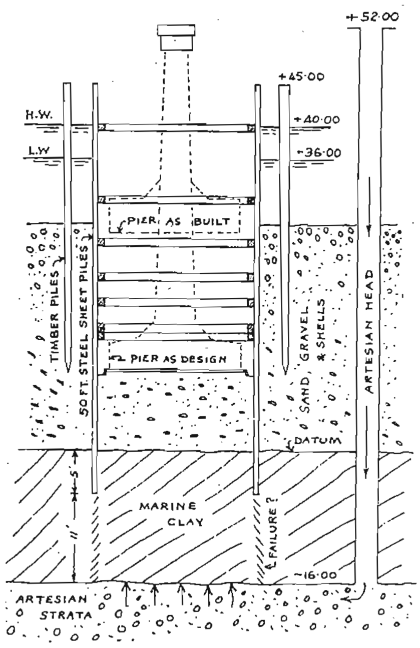


Fig. 11.—Cofferdam to Pier 3.

off. The Wellington City wells adjacent to the bridge site were allowed to discharge freely concurrently with a free discharge from the Lower Hutt City Council wells three-quarters of a mile away. The discharge from each set of wells was about 4 million gallons per day, making 8 million gallons per day total. After four days the drop in the artesian head near the wells was 2 ft. A similar loss of head was noted at the Gear Meat Company's works three-quarters of a mile from the Wellington City wells in the opposite direction from the Hutt City wells.

The inference is that water flows very freely in the artesian measures, and that the volume lost by a large breach in the clay seal would have been enormous. (It also indicates that, if the supply from higher sources were maintained, the drop in pressure at various points in the Hutt Valley might not have been serious.) A gap of 3 in. round the periphery at 3 ft./sec. would have released 120 cusecs or 65 million gallons per day, and such a discharge could certainly not have been sustained by inflow from higher levels.

Levels were taken on the cofferdam until the pier was completed. The settlement was plotted against the weight imposed and the movement corresponded with weight in some sort of fashion until a settlement of $4\frac{1}{2}$ in. had been recorded. The pier moved with the cofferdam. Little further settlement was induced by the placing of some 650 tons of backfill, which made the gross weight (including the cofferdam) 450 tons more than before work commenced. At this stage the cofferdam was 10 in. above original level. It may be assumed that movement of the artesian strata has filled the blister or that friction on the outside of the cofferdam is helping to hold it up. If the latter, a flood might induce settlement. However, it appears more likely that the blister does not consist of water only.

A decision was deferred as long as possible as to the level at which the pier cap should be placed. On the basis of a large-scale drawing of the profile, estimates were made of what might be the least and the greatest settlement of each of the piers, and how a profile could be contrived which would accommodate such movement without looking unsightly. The cap was set 12 in. above true level.

[NOTE.—At the time of writing, very little further settlement has occurred, and the profile of the bridge shows something of a peak at Pier 3. A certain amount of adjustment was made when setting levels for the handrails, kerbs and asphalt road surface.]

3.7. Pier No. 1

Quite apart from the artesian blow on Pier 3, the cofferdam construction had proved to be slow and costly, and pile extraction was proceeding very slowly. On July 3, 1953, therefore, the contractors were instructed to build up the west bank so as to form an artificial island for Pier No. 1 and to construct a caisson of reinforced concrete on the pier site. The caisson was 60 ft. by 20 ft. in plan, with outer walls 3 ft. 0 in. thick at the base, and with three cross walls 2 ft. 6 in. thick, dividing it into four compartments. The height, as first concreted, was 12 ft. 9 in. The concrete walls were designed for possible pressures inwards or outwards, and also with a view to weight for sinking. An offset of 2 in. was formed 2 ft. 9 in. above the cutting edge, and the exterior faces were battered back at $\frac{1}{2}$ in. per foot of height to facilitate sinking. The contractors were very prompt in getting construction under way, and by August 11 the first lift was ready for sinking. Excavation from the interior cells was done by grab, and in six days the caisson was sunk 12 ft. on an even keel. The weight at this stage was approximately 480 tons (260 cu. yd.), so the side friction was 5 cwt./sq. ft. of surface for 12 ft. embedment in sand and gravel.

The caisson was then concreted to a total height of 23 ft. from the cutting edge, giving a volume of 405 cu. yd. and weight of 750 tons. It was sunk a further 11 ft. in 4 days quite readily, the friction at a depth of 23 ft. amounting to a little more than 4 cwt./sq. ft. though the excavation was some 5 ft. below the cutting edge in the middle of each cell.

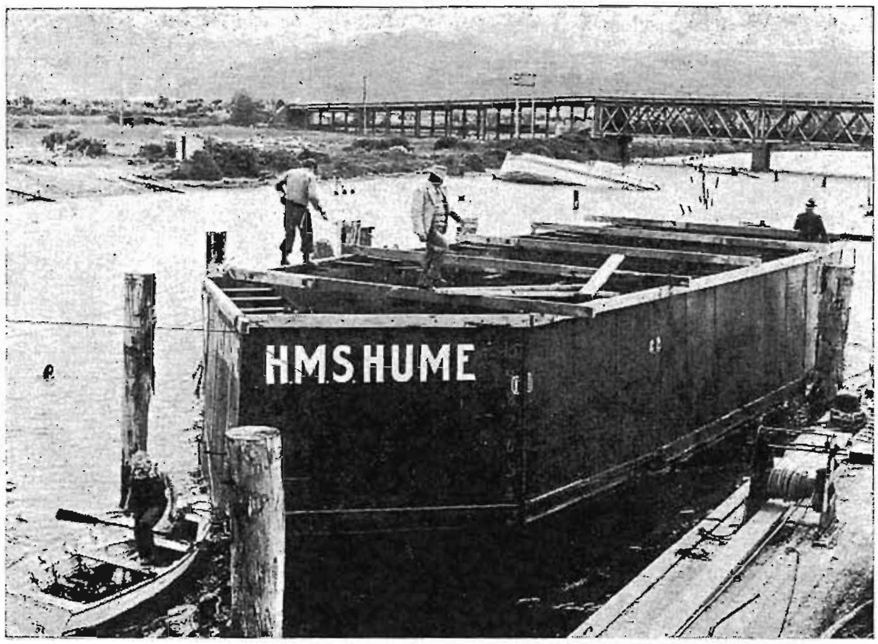
A timber cofferdam was then built up to 11 ft. above the caisson top, and sinking continued. The caisson was sunk 10 ft. in 12 days, but some thin layers of clay had been met, and by the time R.L. 10.5 was reached it had been sunk 2 ft. into what appeared to be the marine clay at a higher level than had been found in the test bore for this site. Grabbing and jetting made little impression on this clay, and after a week's effort it was accepted at this level.

The caisson was then backfilled with a well-graded gravel up to 1½ in., and the pier shaft was built on top. The pier cap was made 2½ in. high, partly to allow for settlement, but mainly to allow for a profile which would suit Pier No. 3.

No floods of any consequence occurred during the sinking of this caisson. The artificial island remained intact, and the caisson finished on an even keel and in the true position.

3.8. Pier No. 2

The caisson construction for Pier No. 1 had proved to be comparatively quick and economical. Some similar process was indicated for Pier No. 2, which is well out into the river. Another artificial island was discussed, and the fill could have been protected by some form of revetment fixed to the timber staging piles already driven round the pier site. However, Mr. Kalaugher proposed the adoption of a floating steel shell formed to the shape of a caisson, as offering the advantage of speed. At this stage Pier No. 2 was very definitely the key to general progress, so the scheme was accepted. Messrs. Hume Industries



Floating caisson used for Pier 2.

(N.Z.) Ltd. tendered for the construction of the floating caisson, which became known as the "tin boat". No restrictions were placed on the company except that the tin boat should be of steel plate, to the outline required, and that it should be capable of withstanding both the internal and external pressures which would be imposed on it. The price quoted was £1,934, which was for about 11 tons of steel in the form of ⅛ in. plate welded complete on the riverbank. The height of the steel shell was to be 12 ft. from the cutting edge. Construction was authorized on Oct. 16, and by Nov. 23 it had been placed in position and filled with concrete.

Sinking proceeded as for the previous caisson, and the concrete was built up to 23 ft. from the cutting edge. Sinking was then resumed, but on January 11, when the cutting edge was at about R.L. 28, or only 5 ft. below the local bed level of R.L. 33, a minor flood occurred which scoured a hole about 8 ft. below bed level, probably accentuated by the proximity of the artificial island. The caisson took a dip upstream with its nose a few inches below the bottom of the scour hole. It was at this stage 4 ft. out of level in an up and downstream direction, but it was subsequently righted and sunk without further incident until the cutting edge reached R.L. 9 as planned on February 5. The flood mentioned was such as, according to the River Board engineer, might occur twice a year. It reached R.L. 46.2 on the gauge at the Ewen bridge and was of the order of 20,000 cusecs.

Subsequent reflection suggests that, if a flood of this magnitude scours locally to 8 ft. below general bed level, the margin of depth of 12.5 ft. below calculated general scour level to R.L. 21.5 is little enough.

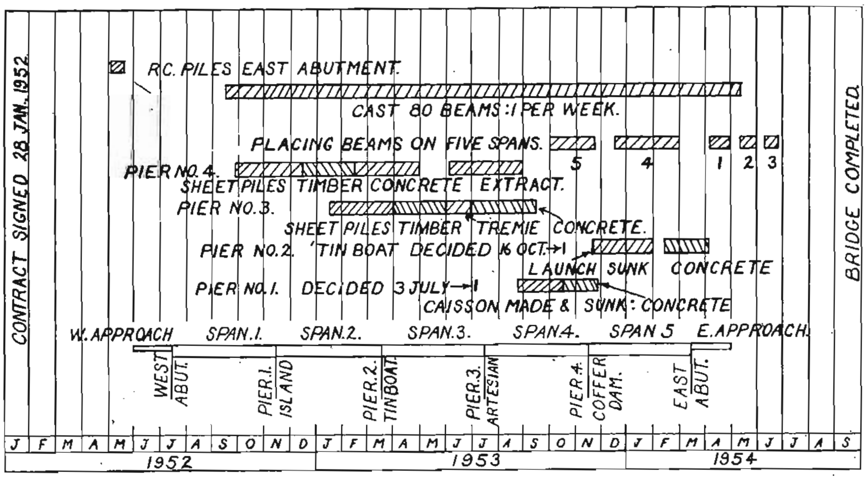


Fig. 12.—Progress chart.

Fortuitously, the margin against scour is, for Piers 3 and 4, far greater than was planned, because of the sheet piles remaining in position. Pier 1 is sunk for 2 ft. into what appears to be the marine clay, and should be amply safe. Pier 2 is therefore the only one which depends on design considerations, and is somewhat better in that the bores show a layer of clay from R.L. 18.0 to R.L. 14.5, and the cutting edge is one foot lower than the designed pier base level. Thus, from the point of view of scour, Pier 2 is appreciably more vulnerable than the others. The "tin boat" has slightly curved sides, and a cutwater at each end.

At the time, naturally, the emphasis was on speed, and the desire to backfill before any artesian pressure could make itself felt.

3.9. Handrails

The handrails were constructed of precast panels 3 in. thick set into grooves in *in situ* posts, and capped with an *in situ* concrete rail. It is not desirable for the handrails to be strained by general deflection, so some degree of latitude was given by leaving the panels free on the vertical ends, and by inserting a ¼ in. thickness of felt between the rail ends and the *in situ* posts. These posts were made to project 1½ in. above the handrail, partly for architectural effect, but mainly in order to render it difficult for a critical eye to detect any deviations in the rail from alignment or profile. The line of the handrails, however, appears to be very true.

3.10. Progress Generally

The progress chart, Fig. 12, shows the principal stages of the work. Pier construction was always the governing factor, and the chart shows what an improvement in time was gained by using caissons instead of cofferdams. Beam-casting could always have been accelerated had pier-construction been faster. The chart emphasizes how rapidly progress was made once the last pier was finished. Beam-placing was slow for the first two spans, but this was due only to concentration on the piers. Had it been expedient, the first two spans could have been placed as rapidly as the last three.

4. CONCRETE

4.1. Preliminary

High-quality concrete is essential in prestressed construction, and, therefore, particular attention had to be given to concrete procedure and tests.

4.2. Stresses

The maximum stress to be catered for is in the bottom flanges of the beams at transfer. It is calculated as 2,200 lb./sq. in. without any loss of prestress, and 1,580 lb. sq. in. after a loss of 15%. Assuming that there is some almost immediate loss of prestress, the maximum compression may, in fact, be about 2,000 lb./sq. in. For a factor of safety of 3 the strength of the concrete should be 6,000 lb./sq. in. The minimum strength aimed at was 6,000 lb./sq. in. cube strength, interpreted as 5,100 lb./sq. in. for 12 in. x 6 in. cylinders. Compared with European practice, these are very modest figures, but for a first venture in prestressed work there seemed to be no point in using very high stresses. The stresses used for the Walnut Lane Bridge in America were somewhat higher, and the quality of concrete very similar.

4.3. Laboratory Tests

The first step in concrete control was to consult M. A. Craven, engineer to the N.Z. Portland Cement Association. Mr. Craven gave general advice to the officers of Certified Concrete Ltd., in whose laboratory the preliminary tests were made. It was decided to adopt a cement content of about 750 lb./cu. yd., and a maximum size of aggregate of ¾ in. to produce a mix which would meet strength requirements, have a not-excessive shrinkage, and be easy to place. A comprehensive report was produced by D. M. Wilson, of Certified Concrete Ltd., giving properties and test results of eleven trial mixes which showed 12 in. x 6 in. cylinder strengths ranging from 5,200 lb./sq. in. to 7,150 lb./sq. in. A typical slump-strength curve is drawn in Fig. 13.

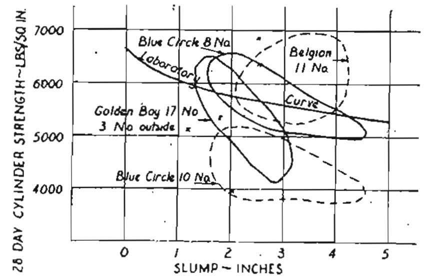


Fig. 13.—Slump in relation to cement brand.

The compressive strengths reached were not particularly impressive, but the aggregate from the Hutt River, which was used throughout, is not intrinsically as hard or as tough as many aggregates in use elsewhere.

4.4. Supply and Control

As a result of the tests, quotations were obtained from Certified Concrete Ltd. for the supply of concrete in agitator trucks, the only stipulations being that the cement content should be 750 lb./cu. yd. in place, that the maximum size of aggregate be ¾ in. and that the minimum crushing strength for laboratory-cured 12 in. x 6 in. cylinders at 28 days should be 5,100 lb./sq. in.

The contractors for the bridge were not necessarily bound to use premixed concrete, and on various occasions in the early stages of the work discussions were held as to the advisability of mixing on site. The advantages would be the elimination of delays and waiting time and more immediate job control. The rate of placing concrete could be attained with the use of quite a small mixer. However, in the actual work, premixed concrete was used throughout for all the high-grade concrete in the beams and diaphragms, and in fact for all the concrete in the bridge except for a small quantity in the deck, kerbs and handrails.

The supply position was at times very difficult in respect of cement and of fine sand, and it was thought that Certified Concrete Ltd. would be in a better position than any other organization to maintain supply. The supply of concrete was thus by Certified Concrete Ltd. on a guarantee of minimum strength. Two sizes of coarse aggregate, one part of coarse sand and a small proportion of fine beach sand were used. The cement and all aggregates were weigh-batched and water was regulated by the mixing effort as read on an ammeter. Under the guarantee, no minimum slump could be specified rigidly, but 1 in. slump was stated as the optimum and 2 in. as the maximum permissible. The slump tests were thus a measure of immediate control; otherwise there would have been no check prior to the routine 28-day tests (and a certain number of 7-day tests in the early stages). Three test cylinders were taken from each beam throughout the work and the results are commented upon later. In the early stages of the beam-casting some truckloads of concrete were rejected owing to the slump exceeding 2 in., but H. W. Cormack, general manager of Certified Concrete, made urgent representations that slump was not a very vital factor, in view of the high cement content and controlled grading, and the requirements as to slump were slightly relaxed. Of 63 slump records for beams, 7 exceed 3 in.

Certified Concrete Ltd. were, under the agreement described, regarded as responsible agents rather than simply as merchants, and the arrangement worked very well in most respects. Some criticism may be made of the fact that prices were quoted for "plastic and unhardened volume at the time of discharge from the delivery truck". The company will not concede that this can be measured in the formwork as compacted concrete. How, then, can it be measured, or what check is there on how much remains in the truck after discharge? It was found that a beam mould made to very accurate dimensions and of a calculated capacity of 18.4 cu. yd. required a nominal quantity of "plastic" concrete 21.2 cu. yd. to fill it. A more predictable basis of measurement would be preferable.

4.5. Placing

Placing of the beam concrete in the initial stages did not proceed smoothly, since the chute from the agitator truck could not discharge at a proper angle into the beam forms. With the chute too flat, the concrete would not flow, and raising the truck on ramps to a suitable level was a cumbersome business. Even when this was done as a trial, the tendency was for the workmen to allow too much concrete to be deposited at one point, thus making proper vibration and compaction impossible. Part of one beam was concreted by using a skip from a 10 R.B., but this method was far too expensive and suffered the disadvantage referred to above of too much concrete in one place. A dump truck was given a trial instead of an agitator truck, but was no better. The final method used for practically all the beams was discharge by chute from the agitator truck into an open-ended steel trough set about 3 ft. above the ground. From this trough the concrete was delivered to the beam by long-handled shovel. With a placing rate of 3 to 4 cu. yd./hr. two men could do the work. This method enabled the foreman to control and direct the placing of concrete and to co-ordinate the vibration and ramming.

4.6. Vibration

Vibration of the concrete was by means of external vibrators fixed to the formwork by steel clamps, supplemented by a poker vibrator applied to the concrete in the usual way. The external vibrators were Consolidated Pneumatic Co. electric vibrators type V-2, rated at 3,000 vibrations per minute and of 220 watts capacity. Eight vibrators were used, giving a total capacity of 1.8 kW. or 2.4 h.p. The vibrating effort was thus $\frac{3}{4}$ h.p. per cubic yard per hour. The concrete was placed in each compartment between diaphragms, the "box" so formed being about 12 ft. long and containing 1.84 cu. yd. (The end block sections contained 3.67 cu. yd., but naturally concrete placing was easier.) The vibrators were fixed first near the bottom of the beam mould, and moved up as concreting progressed. Owing to the need for speedy transfer up the mould and to the next compartment, it was found necessary to use 20 clamps. These took perhaps five minutes to fix, whereas a vibrator could be changed to a new clamp in a matter of seconds.

At the beginning of the work the beam sides were bolted to the concrete beam bases, and the side moulds were thus fixed at the ends (*i.e.*, diaphragms) and bottom to comparatively inert masses. Later, owing mainly to grout leakages, a false timber floor was used, with a length of $\frac{3}{8}$ in. manila rope set in a groove in the timber. This effectively sealed grout leaks at the base and appeared to make the vibration more effective.

4.7. Curing

The importance of curing was appreciated particularly on the thin $6\frac{1}{2}$ in. beam webs. The beams were covered with wet hessian, held in place by rather primitive wooden frames, but the shape of the beams made it difficult to keep the hessian against the webs, and in hot, windy weather the curing was not as thorough as it should have been. It might have been preferable to spray the beams with a sealing compound, but the cost was a deterrent.

4.8. Modulus of Elasticity

(i) Two test cylinders were tested at the Dominion Physical Laboratory of the Department of Scientific and Industrial Research. One cylinder was made with Golden Bay cement and the other with Blue Circle. The results were very similar, being approximately 4,600,000 lb./sq. in. The crushing strength of the cylinders was 6,730 lb./sq. in.

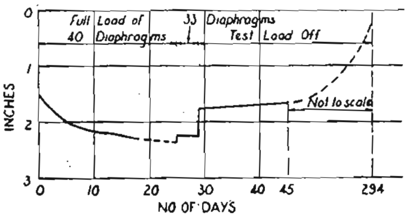


Fig. 14.—Test beam deflection.

(ii) The immediate deflections of the beam under the test load described elsewhere gave a calculated modulus of 4,600,000 lb./sq. in. which agrees very closely with the cylinder tests.

(iii) The "hogging" of the beams as they were stressed provided another check on the modulus. For a tension in the wires of 120,000 lb./sq. in. and a modulus of 4,600,000 lb./sq. in., the "hogging" should be 1.2 in. Measurements were taken of all beams and were fairly consistent, ranging from 1.2 in. to 1.6 in. At this stage the friction of the beam ends on the bed would tend to prevent free movement.

Based on the three methods outlined above, the modulus seems to be well established at about 4,600,000 lb./sq. in.

4.9. Creep

Shrinkage prior to stressing was largely eliminated at the time of stressing. Any subsequent shrinkage would cause the beams to sag. In fact, "hogging" increased during handling, and by the time the beams were placed on the piers the "hogging" was in some cases $2\frac{1}{2}$ in., and varied sometimes by as much as 1 in. from one beam to the next, though the effect was obscured by various inaccuracies in the dimensions of the beams and in the level at which they were set in position. The evidence, however, is that the first obvious effect of creep is a shortening of the more highly stressed lower flange and an increase in hogging. Whether subsequent creep will cause further movement of this nature remains to be seen.

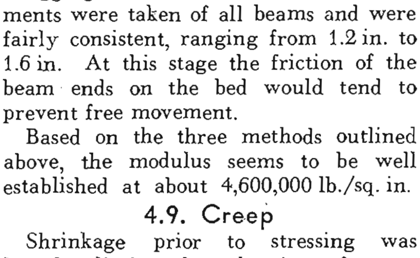


Fig. 15.—Number of tests below strength noted.

The effect of the dead load of the deck, asphalt, kerbs and handrail, and of live load, will, of course, be to cause the beams to sag. Observations will be made of the bridge in service.

4.10. Test Load

The beam under test showed a deflection (see Fig. 14) of 1.5 in. on the application of load and a gradual further deflection with time which reached a maximum of 2.30 in. in 18 days. On the 25th day 7 diaphragms were removed, and the residual deflection measured as 2.28 in. It is unfortunate that no reading was taken on the 25th day before these diaphragms were removed, and the dotted line from 18 to 25 days on the graph is an assumption. Even so, the shape of the curve suggests that deflection with time would not have proceeded beyond 2.5 in. On the 29th day all the superimposed load was removed and the residual deflection measured as 1.76 in. From 29 days to 294 days the deflection gradually decreased to 0.25 in. or 0.30 in. at a fairly constant rate as checked by the observation on the 45th day.

Thus when the load was imposed the immediate deflection was 1.5 in., but when the load was removed the immediate restoration was only 0.6 in. A possible inference as regards creep is that the imposition of load involved a relief of the high compressive stresses of the bottom flange and that this process was faster than the reimposition of those stresses when the load was removed. The change in stress for the top flange was substantially smaller. This explanation may not be completely satisfying, but the point is of some interest.

The test was designed so that under unfavourable conditions cracks might be opened up in the bottom flange. No cracks could be discerned. The loading for the test should have been substantially heavier for it to be of much interest technically, but the test gave a greater degree of confidence to the men who had to handle the beams. A test to destruction would have been much more valuable technically.

4.11. Tests of Crushing Strength and Density

(i) Three test cylinders 12 in. x 6 in. were made from each beam, and the routine was to have them kept in damp sand for 3 days and then laboratory-cured for 25 days. As a criterion for reference, typical preliminary laboratory specimens gave the following results (all 750 lb. cement/cu. yd.):

Mix No.	Slump (in.)	Density (lb./cu. ft.)	Strength (lb./sq. in.)
X_1	0.37	150.2	6,480
Y_2	1.00	150.9	7,153
Z_2	2.25	149.5	6,360
X_3	1.75	149.4	5,845

(ii) The crushing strength showed considerable variation, as will be seen in Fig. 15, on which is plotted the number of tests below a stated strength. Thus 20 tests out of 82 tests were below the guaranteed minimum strength of 5,100 lb./sq. in. Figure 15 also shows a mean strength according to the sequence of casting—namely, the mean for the first 20 beams was 5,980 lb./sq. in. and the mean for the last 26 beams was 4,700 lb./sq. in. (In every case the figure referred to is the mean of the three for any one beam.)

(iii) A first impression from this graph is that the plant operator took less care as time went on and that control should have been more strict. It is submitted that this explanation is not correct. From time to time, low results were obtained, followed by a return to normal. Towards the end of the work—namely, for beams 73 to 82—results were uniformly below the usual standard (except for beams 80 and 81), but since these beams were cast in six weeks, and test results were not available until four weeks after casting, beam production was completed before any effective investigation could be made. The explanation was believed to lie in the use of a certain shipment of Blue Circle cement. When the matter was referred to Mr. Cormack, he confirmed that strengths from this shipment were consistently low, but that he had some evidence that the strength would improve over a longer period than 28 days. Even so, for five beams for which job tests gave a mean strength of 4,290 lb./sq. in., the plant tests (made by Certified Concrete Ltd.) gave a mean strength of 5,030 lb./sq. in.

(iv) The tests referred to above showed better results at the plant than on the job. The same discrepancy appeared in the early stages of the work when 10 tests of concrete cast from May 23 to July 11, 1952, gave a mean strength of 7,200 lb./sq. in. at the plant and 5,450 lb./sq. in. on the job. There is no evidence as to the cause, which might be due to deterioration in transit or to differences in the preparation of specimens.

(v) While the cement may be the cause of the low strengths referred to, it is significant that the low strength tests showed also low density (see Fig. 16). Results are included for some specimens from the piers and abutments which contained only 560 lb. of cement per cu. yd., and these fall into their place on the graph fairly well. So also do the tests marked *R*, which are of concrete made from Ruamahanga aggregates produced at Masterton.

The results marked R_2 are also of Ruamahanga aggregate, using old Australian cement. Here there is no doubt that the cement was the cause of the

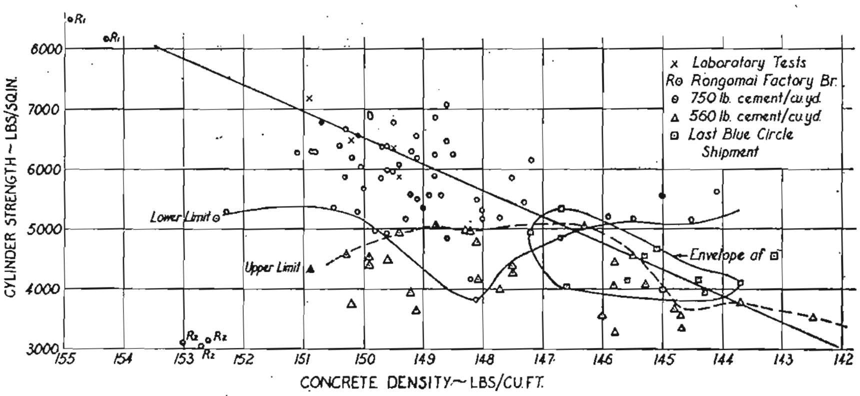


Fig. 16

low strength, since the tests are well below the strength which might be expected for their density. On the other hand, as shown by the envelope on Fig. 16, the tests from the last shipment of Blue Circle cement as used on the Hutt Estuary Bridge show a strength corresponding with their density, and therefore it would appear as if grading was at fault.

Figure 16 suggests that site measurements of density would be as good a check as crushing tests, and the result would be available at once, instead of after 28 days.

(vi) Slump as a measure of strength cannot, on the tests recorded, be made to mean anything unless the type of cement is taken into account. Figure 13 shows slump v. strength for Golden Bay, Belgian and Blue Circle cements. The Belgian cement was used for beams 29 to 37 continuously. Golden Bay and Blue Circle were used at all stages of the beam production. The last shipment of Blue Circle is shown in the lower envelope and was used for beams 73 to 82.

(vii) Figure 17 shows the number of tests which had a stated strength. No reasonable curve can be drawn to represent the result. Many papers are written on what results should be obtained, but not so many on what have been attained.

On November 6, 1953, D. Haliburton, the resident engineer, made an analysis of test results and reported a mean of 6,151 lb./sq. in. with a standard deviation of 630 lb./sq. in. and a coefficient

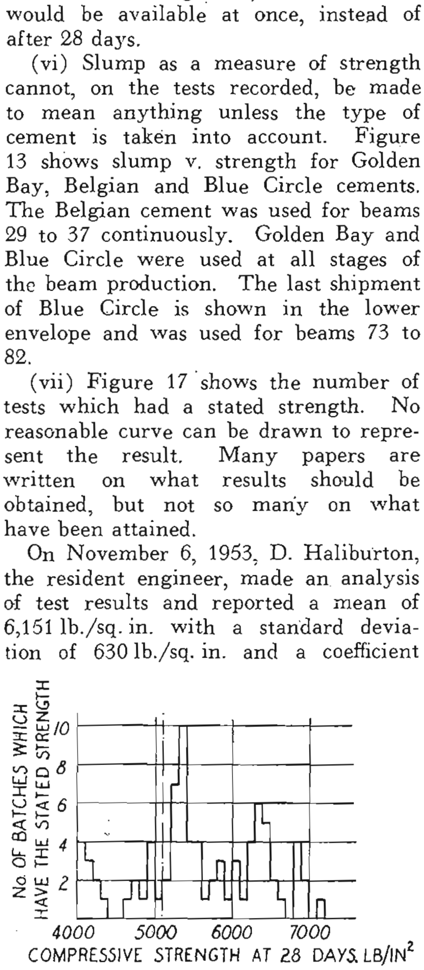


Fig. 17

of variation of 10.2. This was, however, prior to the poor test results in the later stages of the beam casting.

(viii) The comments above all relate to concrete prior to its being placed in the form. It is at this stage that the human element is most important, and careless workmanship can vitiate the results of the most painstaking mix design. The only evidence available on this score is a specimen from one of the broken beams. This was ground to a 5 in. cube and crushed at 6,800 lb./sq. in., corresponding to a 12 in. x 6 in. cylinder strength of 5,900 lb./sq. in. The cylinders made during the casting of this beam averaged 5,200 lb./sq. in. As far as it goes, the test is reassuring.

4.12. Broad Conclusions

These are that the quality of the cement and the method of preparing specimens have a much greater effect on test results than is commonly appreciated; also that field density tests should be used in conjunction with slump tests as a measure of job control. The suggestion that the concrete deteriorates in transit by agitator truck should be investigated, and no doubt will be.

Another intriguing point is: 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?

An advance copy of this paper has been sent to Certified Concrete Ltd., so that their comments may appear in the same issue of *N.Z. Engineering* (see p. 375).

5. COSTS

5.1. General

Particular interest attaches to the cost of this bridge, partly because of the prestressed construction and partly on account of the nature of the contract. Costs as given in various tables are not quite final, since this paper is being written before the bridge is completed. They are, however, substantially correct. The costs as given are meant to present information in such form as will be most useful to engineers, and therefore the labour costs of the prestressed work have been given in detail.

A list of tables of costs is as under:

I.—Bridge Costs per Square Foot.

II.—Bridge Costs: Percentage of Permanent Material.

III.—Hutt Bridge: Unit Cost of Prestressing.

IV.—Superstructure: (a) 7/75 ft. Spans in Reinforced Concrete. (b) 5/105 ft. Spans in Reinforced Concrete. (c) 5/105 ft. Spans in Prestressed Concrete.

V.—Comparison of Costs: Table IV.

VI.—Tender for Superstructure: Steel Plate Girder.

VII.—Hutt Bridge: Labour Costs of Superstructure.

VIII.—Hutt Bridge: Analysis of Costs.

Attention is drawn to the comparatively small proportion of the cost of bridges generally, which is made up of permanent materials as delivered on site —see Tables II and V. Table II shows that this is not peculiar to the Hutt Estuary Bridge.

The project was commenced with the object of building a permanent bridge in the shortest possible time, and a "target" contract was arranged with selected contractors, Messrs. Wilkins and Davies Construction Co. Ltd., on the basis of preliminary drawings, while certain essential materials were ordered even in advance of this arrangement. No mistakes have been revealed in the design calculations or quantities, but it became apparent after the work had been in progress only a few months that the estimate was very low in respect of labour, plant and temporary materials. In addition to this, reinforcing steel and high-tensile wire had to be purchased at prices about 50% higher than at present. Over the construction period there has been a gradual increase in the price of nearly all materials, and there have been substantial increases in labour rates.

No matter what explanations may be offered, the cost of the work has caused much concern, and the costs submitted in this paper are intended to demonstrate (a) that the adoption of a prestressed superstructure was not merely an interesting experiment at the expense of local bodies, and (b) that under present-day conditions the overall cost is reasonable.

It may be argued that tenders should have been called. If tenders had been called, a figure would have been available against which to check actual costs. However, the preparation of complete contract documents would have taken

TABLE I.—BRIDGE COSTS PER SQUARE FOOT.

Cost in £ per sq. ft. overall—not including approaches or design charges.

No.	Name		Date	Sq. ft.	Cost
1.	Kelburn Viaduct (Wgtn., N.Z.)	Cost	1928	7,100	3.1
2.	Princes St. Bridge (Ch'ch, N.Z.)	Cost	1931	2,440	1.6
3.	Daggs Bridge* (N.Z.)	Estimate	1945	4,600	1.5
		Tender	1947		1.9
		Tender	1951		4.4
4.	Te Hoe Bridge (N.Z.)	Estimate	1949	2,800	2.1
		Tender	1953		4.6
5.	Wanganui* (N.Z.)	Tender	1953	33,400	10.2
6.	Mangahao Bridge (N.Z.)	Tender	1953	5,200	5.0
7.	Walnut Lane (U.S.A.)	Tender	1949	19,400	11.0
8.	Scalyn (Belgium)	Tender	1948	14,400	4.0
9.	Freyssinet Arches (3) (Venezuela)	Tender	1952	176,500	11.0
10.	Hindiya (Iraq)	Cost	1953	25,600	13.7
11.	Petone Overbridge (N.Z.)	Cost	1951	15,000	3.3
12.	Hutt Estuary (N.Z.)	Cost	1952–4	31,400	8.6

1. Monumental reinforced concrete: on land.
2. Reinforced concrete: piled foundations.
3. Reinforced concrete. (* Not built.)
4. Flat slab spans—30 ft., 40 ft., 30 ft.
5. Caisson foundations; steel plate girders, concrete deck. (* Not built.) Adjusted for existing foundations.
6. Piled foundations; composite joist and concrete deck; spans 60 ft., 80 ft., 60 ft.
7. Prestressed spans, 74 ft., 160 ft., 74 ft., on land.
8. Prestressed continuous 410 ft.; two spans; on existing abutments and central pier.
9. For three similar viaducts; on land; at $2.83 to £1; central arches, 500 ft. span.
10. See *British Constructional Engineer*, June, 1954.
11. Skew spans reinforced concrete; on land.

TABLE II.—BRIDGE COSTS: GENERAL.

Percentage of Permanent Material

The following table gives the percentage of total contract cost chargeable to permanent material as supplied on site —namely, in the bridges under review, to concrete, reinforcing steel and structural steel.

1.	Wanganui Bridge: As tendered 1953 (not including structural steel girders and therefore comprising piers, abutments and decking)	13
1a.	Wanganui Bridge: As tendered overall	32
2.	Princes Street Bridge, Christchurch, 1931: Reinforced concrete on piles	22.5
3.	Carlton Bridge, Christchurch, 1929: Reinforced concrete on piles	30
4.	Mangakino Bridge, 1947-49: Concrete girders on tall piers	29
5.	Maraetai Powerhouse: 225 ft. concrete arch on rock	32
6.	Mangaomeka—East Coast: Steel girders on piles	26
7.	Hutt Bridge: Pier No. 4 only	11
8.	Hutt Estuary Bridge: Total	26.5

Note: Nos. 4, 5 and 6 by courtesy of Ministry of works.

TABLE III.—UNIT COST OF PRESTRESSING LABOUR
(Main Beams : Per Beam)

		Standard	Hutt Average	Hutt *
(a)	Place cores, extract and clean. 1.35 m-h. for 10 ft., on 420 ft. = 57 m-h.	£20 0 0	£22 10 0	£17 2 0
(b)	Make up 24 wire cables. 2,220 lb. at 35 lb./m-h. = 64 m-h.	22 8 0	14 8 0	5 17 0
(c)	Place 24 wire cables. 2,220 lb. at 75 lb./m-h. = 30 m-h.	10 10 0	5 4 0	1 16 0
(d)	Tension 24 wire cables. 48 pairs at 0.4 = 19 m-h.	6 13 0	20 14 0	8 14 0
	Total:	£59 11 0	£62 16 0	£33 9 0

Comparison with standard Magnel rates as given by C.A.C.A. T.C.47 1952 (at 7s. per man-hour).
Hutt * shows the rates for 30 beams made during the 6 weeks ending February 28, 1954, namely, in full production towards the end of the work and in summer time. (Eighty beams in the works.)

time and caused the postponement of the work for perhaps a year. **Further,** no contractors in this country have had much experience in prestressed construction, and tenders would almost certainly have been high. Again, if contractors are compelled to cover flood risk in their tenders the public authority may have to pay for floods which do not occur. More commonly, perhaps, the contractor is persuaded to reduce his price and then makes a claim later for hardship due to flood damage or losses. In general, the writer favours the time-honoured method of calling tenders, but points out that, for bridges involving a substantial flood risk, quite apart from a new type of design, the problem is not as simple as it may appear.

The day is long past when, as in Christchurch in 1931, the writer prepared an estimate for a small bridge for the city engineer at £4,200. Nine tenders were received, ranging from £3,573 to £4,473. Four were below the estimate and five were above it.

On the broadest basis, the number of properly equipped bridge contractors in New Zealand is not nearly enough to build or rebuild the country's bridges. It seems a fair argument that, if bridge contracting were a highly profitable business, there would, owing to the

TABLE IV.—COST OF SUPERSTRUCTURE

(a) *Seven 75 ft. Spans in Reinforced Concrete (Estimate)*

1. *Labour*:	
Structural steel: 5 tons @ £50	250
	£
Reinforcing: 196 tons @ £25	4,900
Concrete: 2,150 **cu. yd. @ £2**	**4,300**
Formwork: 64,000 sq. ft. @ 4s.	12,800
Staging (3 access bridge equiv.)	12,000
Total:	34,250
Insurances, holiday pay @ 8%	2,750
Total Labour:	37,000
2. *Permanent Material*:	
Structural Steel: 5 tons @ £100	500
Reinforcing: 196 tns @ £75	15,900
Concrete: 2,150 cu. yd. @ £6	12,900
Total:	28,400
3. *Temporary Material*:	
Formwork: 64,000 sq. ft. @ 1s. 6d.	4,800
Staging	6,000
Sundries	2,000
Total:	12,800
4. *Plant*:	
Mostly on staging	8,000
5. *General*	
Total to here:	86,200
Wet time, cartage, supervision and overhead at 15 %	12,800
Total:	99,000

Note: This design would require two more piers.

(b) *Five 105 ft. Spans in Reinforced Concrete (Estimate)*

	£
Structural steel: 5 tons @ £50	250
Reinforcing: 312 tons @ £25	7,800
Concrete: 2,650 cu. yd. @ £2	5,300
Formwork: 72,000 sq. ft. @ 4s.	14,400
Staging	12,000
	39,750
Insurances, holiday pay at 8%	3,150
Total Labour:	42,900
Structural steel: 5 tons @ £100	500
Reinforcing: 312 tons @ £75	23,400
Concrete: 2,650 cu. yd. @ £6	15,900
Total:	39,800
Formwork: 72,000 sq. ft. @ 1s. 6d.	5,400
Staging	7,000
Sundries	2,000
Totals	14,400
Mostly on staging	8,000
Total to here:	105,100
Wet time, cartage, supervision and overhead at 15%	15,900
Total:	121,000

Note: This design would require slightly heavier piers.

(c) *Five 105 ft. Spans in Prestressed Concrete (Actual)*

	£
Net as Table VII	29,560
Insurances, holiday pay at 8%	2,360
Total Labour:	31,920
High-grade concrete: 1,770 cu. yd. @ £7 5s.	12,850
Reinforcing steel: 40 tons @ £75	3,000
Ordinary concrete: 50 cu. yd. @ £6	300
Grout	800
Roller bearings and expansion joints	1,150
Lee McCall transverse rods	1,810
High-tensile wire, 0.276 in.: 83 tons @ £152	**12,600**
Magnel anchors and wedges	6,370
Frames for rubber cores	670
Total:	39,550
Concrete waste and waiting time, steel waste and offcuts	1,220
Total:	40,770
Casting beds	2,000
Beam moulds	4,000
Beam handling	4,500
Rubber cores	1,500
Formwork and sundry	2,000
Total:	14,000
For beams: casting, to stock and to bridge	6,900
Total to here:	93,590
Wet time, cartage, supervision and overhead at 15%	14,000
Total:	107,590

operation of the law of supply and demand, be more firms engaged in it. One of the difficulties is that expensive and specialized plant is necessary. The suggestion is offered that a company might be formed to hold a pool of plant which could be hired to contractors (as is done in England) or that the Government might operate such a pool.

The question of plant is most important, since workmen are more and more reluctant to do heavy work which can better be done by plant. Even in England during the depression the writer had evidence of this. It follows that, if the trend is (and on social grounds it must be) towards the use of more and more plant, then work must be better organized and plant must be made to work longer hours.

5.2. Unit Costs

Figures of costs can be made to prove anything, depending on how an analysis is made and how much is booked to "sundry labour". The figures given in the tables should make clear what is included in the unit costs and what is not. Owing to the nature of the work, it is not easy to make comparisons with ordinary reinforced concrete work or with other bridges. The labour cost for bending and fixing reinforcing was in general £25 per ton. Formwork labour per square foot was 3s. 10d. for abutments and 2s. for the approach spans as offering the best comparison with normal practice. Concrete was mostly delivered in agitator trucks. The cost of placing and ramming or vibrating per cubic yard was 33s. for the precast beams, all actually placed by long-handled shovel, 30s. for the approach spans and from 4s. 7d. to 10s. 6d. for the piers. The low figures for the piers are for concrete which was largely handled by chutes from agitator trucks, or by crane skips. Comparisons can be misleading, but it is believed that the labour output per man-hour on formwork, reinforcing steel and concrete was very much the same as on five bridges built by the writer in Christchurch from 1929 to 1932 in so far as conditions were similar. Particularly during the latter half of the work, the contractors were able to command quite a good class of labour, much better than is available in many of the more remote parts of the country.

For the special operations relating to prestressing, the best guide to performance is given in Table III, which compares costs on the bridge with typical figures published by the Cement and Concrete Association (England). This table shows that the bridge costs per beam are very close to typical costs, though individual items vary. The making up and placing of wires was better than standard, but the cost of actual stressing was more. This discrepancy is probably due to differences

TABLE V.—COMPARISON OF SUPERSTRUCTURE COSTS.

(Condensed from Table IV.)

	(a) Reinforced Concrete 7/75 ft.	(b) Reinforced Concrete 5/105 ft.	(c) 5/105 ft.
1. Labour	£37,000	£42,900	£31,920
2. Permanent material	28,400	39,800	40,770
3. Temporary material	12,800	14,400	14,000
4. Plant	8,000	8,000	6,900
5. Overhead, etc.	12,800	15,900	14,000
Total	£99,000	£121,000	£107,590
Add 2 extra piers, say	35,000		
Add for heavier piers, say		3,000	
Total	£134,000	£124,000	£107,590
Figures are for 525 ft. x 53 ft. 9 in. = 28,200 sq. ft.			
As above: Cost per sq. ft.	£4.7	£4.4	£3.8

Notes:

1. Table VI for Wanganui tender shows £4.78 per sq. ft., but this includes contractors' profit, which is not included in Table V.

2. Actual costs for prestressed spans include probably more waste and sundries than have been allowed for in estimates for reinforced concrete.

3. Some proportion of access bridge costs should perhaps be charged to the prestressed construction.

4. Costs for prestressed spans include about £2,000 for broken beams and initial changes of method: also £4,300 for cost of high-tensile wire at £152 instead of, say, £100 per ton.

TABLE VI.—TENDER FOR SUPERSTRUCTURE: STEEL PLATE GIRDER

Based on Wanganui Bridge tenders, 1953. For five 115 ft. spans = 575 ft.; overall width 46 ft. 10 in. Area of superstructure 26,900 sq. ft.

		Sq. ft.
1. Structural Steel: 604 tons @ £154	£93,000	£3.46
2. Formwork: 42,000 sq. ft. @ 5s. 3d.	11,000	0.41
3. Reinforcing: 54 tons @ £90	4,860	0.18
4. Concrete: 635 cu. yd. @ £12	7,620	0.28
5. Plant and sundry, say	12,000	0.45
Total:	£128,480	£4.78

Notes:

1. Plant is estimated from tenders for whole work; may be high or low, but probably within £3,000 either way of a true allocation. (Tenders not accepted).

2. Price of £154 for structural steel is made up approximately as under. No duty is included.

England:	
Supply to works	£43
Fabrication	40
Shot blast	11
Two coats red lead	6
Total f.o.b.	**£100**
Transport:	
Sea freight	13
Local transport and offload	8
Cost on site	£121
On site:	
Assemble	10
Erect	15
Two colour coats	8
Total contract price:	£154

in timekeeping. The figures for 30 beams in good weather towards the end of the work show a cost of little more than half the average, thus emphasizing the effect of experience with a trained team of workmen.

The cost of setting up and stripping forms for the beams, shown in Table VII as 1s. 1½d. per square foot, is higher than was anticipated. Much—or, indeed, most—of the cost of setting up was due to the need to engage the devices which held the rubber cores in place. The trend in America—and, to some extent, on the Continent—is to place the cables in sheaths which are concreted in permanently, and this method may well be more economical overall.

5.3. Piers

Pier costs as shown on Table VIII demonstrate that the cofferdam method was a very expensive way of building a pier. For Pier No. 4 (the first one built) the cost of concrete and reinforcing steel delivered to the site was £3,195. The cost of building the pier on dry land would have been about £5,100. The cost in Table VIII is £26,318. Yet the sheet pile cofferdam was "closed" without delay and without resort to tapered or other special piles, and there were no special difficulties with the timbering or pumping. De-watering was, in fact, very easy, owing to the good seal provided by the clay layer. The driving and extraction of the sheet piles would have been much simpler had heavy floating plant been available. Under the circumstances the decision to adopt caisson construction for the third and fourth piers (numbered 1 and 2) was amply justified by costs. Now that the work is completed, the decision seems obvious, but the caisson method required the acceptance of a very much greater flood risk for a brief period than did the cofferdams.

6. ACKNOWLEDGMENTS

The contractors were the Wilkins and Davies Construction Co. Ltd., whose

TABLE VII.—LABOUR COSTS FOR SUPERSTRUCTURE.

Net costs based on nominal rate of approximately 5s. 6d. per hour and effective rate of approximately 7s. per hour. Subject to addition of 8% for insurances and holiday pay, and to 15% for supervision, overhead, wet time and travelling.

	£	s.	d.	
1. Installation of beds and moulds for casting beams and diaphragms				£3,262
2. Making beams and diaphragms:				
(a) Diaphragms: per set of 7 No.				
Set up and strip forms: 75 sq. ft. @ 1s. 4d	£5	0	0	
Place and strip core holes: Item	3	13	0	
Concrete: 2.75 cu. yd. @ 40s.	5	9	0	
Lift to stock: 7 No. @ 8s. 9d.	3	0	0	
Place on beds: 7 No. @ 8s. 9d.	3	0	0	
Total:	£20	2	0	
(b) Making beams: each				
Set up and strip forms: 1,250 sq. ft. @ 1s. 1½d.	£70	0	0	
Concrete (premix): 18.5 cu. yd. @ 33s.	30	6	0	
Steel bend and fix: 0.4 ton @ £33	13	2	0	
Total:	£113	8	0	
(c) Prestressing				
Place cores, extract, clean 4 No. @ £5 12s.	£22	10	0	
Bed anchor plates: 8 No. @ 7s.	2	14	0	
Make up 24 wire cables: 440ft. @ 8d.	14	8	0	
Place 24 wire cables: 440 ft. @ 3d.	5	4	0	
Tension 24 wire cables: 4 No. @ £5 3s.	20	14	0	
Grout cores: 0.8 cu. yd. @ £10	7	14	0	
Total:	£73	4	0	
Total per beam	£206	14	0	
For 80 beams				£16,616
3. Handling beams: 80 No. @ £75				5,980
4. Transverse prestressing and completion of superstructure				3,702
				£29,560

TABLE VIII.—ANALYSIS OF COSTS.

Figures in £

	Material	Labour	Plant	Total	%
A:					
Pier No. 1: "Island"	7,455	2,965	3,661	14,081	6.6
Pier No. 2: "Tin Boat"	7,507	2,496	3,267	13,270	6.2
Pier No. 3: "Artesian"	13,971*	6,190	5,305	25,466	12.0
Pier No. 4: "Cofferdam"	11,723*	7,824	6,771	26,318	12.4
Abutments and piles	6,190	1,644	885	8,719	4.1
Access bridge	7,782*	5,221	3,964	16,967	8.0
Total substructure	54,628	26,340	23,853	104,821	49.3
Percentages (across)	52	25	23	100	
B:					
Superstructure	54,770*	29,560	6,900	91,230	43.0
Kerbs and handrails	4,028	2,890	227	7,145	3.4
Approach spans	3,040	868	93	4,001	1.9
Asphalt	5,000	—	—	5,000	2.4
Total superstructure, etc.	66,838	33,318	7,220	107,376	50.7
Percentages (across)	62	31	7	100	
Total to here	121,466	59,658	31,073	212,197	100.0
Unallocated	7,266	17,725	6,079	31,070	14.6
Total to here	128,732	77,383	37,152	243,267	114.6
Percentages	53.2	31.5	15.3	100.0	

Notes:

1. Salvage temporary material allowed.*

2. To the total as given above of — £243,267
   add contractors' site supervision — 5,804
   and contractors' office overhead — 7,200
   Total: £256,271

3. Unallocated comprises mostly:
   Material: Sheds, power, rent, water, telephone.
   Labour: Includes
   Insurances and holiday pay — £7,000
   Wet time and travelling — £3,000
   Cost of camp — £5,000
   Plant: Includes sundry cartages — £3,000
   and petrol, oil, fuel — £2,000

4.	Group A	Group B	Total
Permanent material	£18,775	£49,675	£68,450
Percentage of allocated cost	18	46.5	32.0
Percentage of total cost	15	38.5	26.5

co-operation at all stages is acknowledged. In particular, mention must be made of the foreman in charge of beam-placing, who said, on the occasion of a very informal ceremony when the last beam had been placed, "This has been not just a job; it has been an adventure and a challenge."

Also to: Don Fraser, subcontractor for pile-driving; Hume Industries Ltd., for the "tin boat"; Wellington city engineer, for asphalt paving; D. Haliburton, resident engineer; G. Cooper, site representative for consulting engineer; F. E. Greenish, for architectural advice on proportions and general effect of piers, abutments and handrails within the limits dictated by structural necessity; the technical committee, as mentioned in the introduction; and to the representatives of the five local bodies on the joint committee, for being willing to authorize a new type of construction, and for their close interest in the progress of the work.

APPENDIX

Comments on Concrete Mixes

H. W. CORMACK, B.Sc., A.M.I.C.E.*
(Associate Member)

Preliminary Tests

MR. MORRISON states that the preliminary compressive tests were not particularly impressive, and attributes this to the Hutt River aggregate. While it is true that the particles in Hutt River aggregate are traversed by joint planes which tend to be weathered, it is my experience that this is not a significant factor in concrete strengths in Wellington until strengths are of the order of 8,000 lb./sq. in. The trial mixes showed that a water/cement ratio of 0.42 could be attained at a workable consistency with a cement content of 750 lb./cu. yd. and the preliminary tests at water/cement ratio approximating that gave values not departing greatly from 6,000 lb./sq. in. This is right on the "dot" for the water/cement ratio curve for Golden Bay cement—the cement used in the trial mixes. It is my opinion that the preliminary tests behaved very much as they should have.

It is true that two sets of mixes gave results of 7,150 lb./sq. in. and 5,150 lb./sq. in., but the former was of such a workability as to be thought undesirable and the latter was at a slump of 5½ in. and was deliberately made to fix one end of the curve of slump versus strength.

Regarding slump of concrete, my recollections, refreshed by recent readings, are somewhat different to that of Mr. Morrison. The recommendation was that it be in the range of 2 in. to 2½ in., and it is possible that, in the very early stages of construction there was some unnecessary strife and turmoil in trying to place concrete drier than this. The layout, vibrators and personnel were just not able to cope with the drier concrete.

Strength Results

Mr. Morrison suggests that there is some difference between plant and job tests, but the amount of this difference is actually not considerable. He mentions Mr. Halliburton's analysis as at November 6, 1953 (when the job was two-thirds done) of a mean 6,150 lb./sq. in. and a coefficient of variation of 10.2%. Plant results at that time gave a mean of 6,550 lb./sq. in. and a coefficient of variation of 10.8%—very similar results considering that the testing was not necessarily from the same batches and quite commendable results which, by universal standards, can be classified as excellent control.

From there, results deteriorated as also did cement supplies, and, while the tests were quite reasonably uniform amongst themselves with a coefficient of variation of 10.5% at the site and 13.8% at the plant, gave average results appreciably lower. The final result is that the mean for the whole of the plant tests was 6,100 lb./sq. in. with a coefficient of variation of 15.7% Almost all of the tests under 5,100 lb./sq. in. occurred in the last third of the job, and many of these tests did not come to hand until after the concrete pouring was completed. There are various factors which there is not time to examine, but the essential point is the one that Mr. Morrison mentions in his broad conclusions—that the quality of the cement has a much greater effect on test results than is commonly appreciated. On this particular work cement was supplied from five different countries. The number of brands used was even more than that, for, in the overseas shipments, cement from different mills is packaged under the same marking, and it is quite possible that cement was obtained from as many as twenty different mills. Not only does each cement have its own strength characteristics, but it can give to the concrete quite different workabilities, and, with variation in cement brand, uniformity of control is most difficult indeed.

* General Manager, Certified Concrete Ltd., Auckland.

Yield

Mr. Morrison talks of yield of concrete, and mentions a figure of 21.2 cu. yd. to fill 18.4 cu. yd. of net, vibrated and compacted concrete. These figures, I suggest, require some amplification. Between the plastic volume of concrete and the set volume in place there is, of course, some decrease, but there are surprisingly few jobs where it is possible to make this appraisal with certainty. Pertinent factors are method of placing or intensity of vibration where this is used, accuracy of moulds and material wasted on ground or over-ordered. The first two factors may, to a certain extent, be considered together, for forms accurate for hand-placing will not necessarily be so under long and intense vibration.

I have taken out the quantities supplied for the last thirty-five beams poured and find that the average quantity supplied was 19.6 cu. yd.—or 6½% greater than the net amount—a figure which, considering that wastage on the ground was not inconsiderable, is, I submit, more than reasonable agreement and one which will be of interest to members.

ANNUAL ELECTIONS

Nomination of Members of Council

Members are reminded that nominations for the election of members of Council must be received at the headquarters of the Institution not less than 60 days before the annual general meeting — that is, in respect of the forthcoming election, by Thursday, December 23, 1954. Nomination forms may be obtained from the secretary or from branch honorary secretaries. The ballot papers will be issued together with the Council's annual report, notice and programme of meeting, not less than 28 days prior to the annual general meeting. The ballot papers must be returned to the secretary, completed, within 14 days. Scrutineers appointed in Wellington, but being neither members of Council nor nominated for election to any office on the Council, will then commence the count.

The results may be reported to the Council immediately on completion and not less than seven days before the annual general meeting, at which they will be announced.