Mr. Toogood said he would like Mr. Jones to write a paper about this subject.

Br. Bell said that in this connection there were some tests carried out at the National Physical Laboratory, the results of which Mr. Toogood might like to see.

Mr. A. Murray said that it seemed that every paper he wrote raised a good deal of argument among members of the conference. It was fortunate for him and he appreciated the point. Mr. McLaren had asked why steel joists were used instead of concrete beams. The reason for this was set out on page 232 of the *Proceedings* of last year: there he would find an illustration of why this was done. In the paper concrete beams were shown as costing £1.82 per square foot of deck area, whereas the concrete slab and rolled steel joists cost £1.38 per square foot of deck area. That was the reason he had used the steel joists. The President had brought up the question of binding wire around the beams which were encased in concrete. That was a point upon which he had joined issue with the Main Highways Board, and from his own experience of bridges built without the binding wire he was satisfied that the practice was sound. He had a number of bridges which were not on main highways and which were not built on grant proposals, and in these joists were encased without the aid of binding wire, and concrete after three years was still adhering to the joists. He had endeavoured to explain in his paper why concreting of the outside joists only was carried out in order to effect a saving in cost. Mr. Turner had asked why he did not reinforce the bottom corners of the culverts as he had done the top slab. He had made a note of that very point in the bottom right hand corner of the table on p. 246c where he said: "On poor foundations, the invert is to be widened outside side-walls to give greater bearing area and is to be reinforced as for top slab." The reinforcement in the invert was not required, as on the top slab. With poor foundation he agreed that Mr. Turner's contention was correct. Mr. Toogood had mentioned two bridges, one a steel truss bridge at Henderson. That bridge was manufactured in America and imported, and was sent to New Zealand nearly twenty years ago. It had been badly neglected and was due for reinforcement at any time. It was interesting in that it was fabricated in another country and imported into this Dominion. In connection with the concrete at Lincoln Bridge, this illustrated the use of poor aggregate. The aggregate there had been the cause of the trouble, as in the case of other bridges. It was due to a poor grade of shingle. As to the point which Mr. Jones had taken

up on his behalf, regarding the making of a stronger job by the use of steel joists, this was true. There was plenty of literature on the subject. These beams had been tested under normal conditions.

Mr. F. J. Jones referred to the 7′ 6″ culvert shown in the picture, and asked what was the depth of the slab in that case.

Mr. A. Murray replied that it was 8½″ reinforced concrete of the ordinary standard. The invert was reinforced in the same manner.

A vote of thanks was accorded to Mr. A. Murray for his paper.

WAIHOU RIVER BRIDGE, KOPU, AND PILED FOUNDATIONS.

By A. J. Baker, A.M.Inst.C.E.

The Waihou River flows into the Firth of Thames, northward, along the eastern side of the rich alluvial valley known as the Hauraki Plains. This fertile area of land has in the last decade or two been the scene of extensive drainage works, and is now a closely farmed prosperous district. Both the Thames Railway, which serves this district, and the Borough of Thames, its chief business centre, are located on the opposite side of the river. Prior to the building of the bridge at Kopu, the nearest structure to the mouth was the bridge situated at Kopu, near Paeroa, 22 miles upstream. So for many years the attention of the district had been directed towards the need for bridging the Waihou river towards its mouth. The growth of road transport and the building of a good metalled road between Pokeno and the Plains, thus providing a route between Auckland and the Thames and the Coromandel Peninsula much shorter than any other possible route, proved important factors in the location of the proposed river crossing.

Over a period of years a number of sites for the bridge had been proposed between Paeroa and the mouth of the river. At some of these, ferries had been worked with more or less success for a time but the inadequacy of these became very apparent with the passing years. Finally the Public Works Department, after a close examination of the merits and demerits of the several possible sites, definitely advised the building of a bridge at Kopu at a site two miles from the mouth. Generally this site suited the trend of business and the flow of traffic admirably, and proved the shortest possible route for the greatest amount of traffic. The decision in favour of the Kopu site has been amply justified now that the bridge is in service and has, it appears evident, proved satisfactory to all parties concerned.

In 1922 negotiations regarding the financing of the cost of the proposed structure which was then estimated at £60,000, took definite shape. The three Local Bodies interested, the Hauraki Plains County Council, the Thames County Council, and the Thames Borough Council, between them agreed to find £30,000 towards the cost of the work, the balance to be found by the Government. This basis of finance was adopted, and it was arranged at the same time that the preparation of plans of the structure and the carrying out of the work should be undertaken by the Public Works Department.

The character of the river is such as to make the work of bridging it in its lower reaches a major problem. In the first place, the river is a navigable stream and is used by steamers of several hundred tons burden, a substantial opening span thus being necessary. In the second place, the river having a definite estuarial character is wide, involving a long bridge, and it was known to have a soft sand and mud bottom, so that piled foundations would be deep.

The first step was the testing of the foundation. A series of 12 test piles was driven across the site, spaced at 100 feet intervals. These were of Kahihatea and Pinus Insignis timber and averaged 1ft. 11in. at butt and 9in. at small end. Driving was done with a 36 cwt. drop hammer, and the penetration in ground averaged 56.5ft. varying from 40ft. to 80ft.; and safe bearing averaged 16.8 tons computed by the "Engineering News" formula. The bed of the river for the first 30ft. of penetration proved very soft and at that depth these piles only averaged by the same formula a safe bearing figure of 4½ tons. However, the rate of increase of safe bearing improved with further driving, and values plotted in terms of penetration after the first 20 to 30 feet generally gave an irregular line approximating a straight line graph. The results indicated that there would be no particular difficulty in constructing a pile foundation, though long piles would have to be used.

Design.

The loading to which the structure had to be designed was the Public Works first-class standard for traffic bridges, this being 16 ton traction engine loading with string of loaded trailers. A prime consideration was the need for the most economical design possible, consistent with permanence, good design features and good appearance. The structure would be an expensive one in any case; and as the financial resources of the districts contributing were limited, it was necessary to strictly aim at economy. The factor of permanence pointed to a reinforced concrete structure throughout. That, however, involved maximum weight and on a soft foundation such as that at the bridge site, any extra weight in the structure would be carried at the expense of considerably greater cost in piling, and in pier construction, or of increased number of piers. After consideration of alternatives it was decided to adopt 60 ft. spans with steel plate girders, and reinforced concrete deck laid and bonded on to the top chord of the plate girders. The design required a perfect bond between steel and concrete, so that portion of the compressive stress in the top chord would be taken by the concrete. Having regard also to the fact that complete rigidity against lateral movement would be provided by the concrete deck bonded to the top chords of the girders, it was possible to design a comparatively slender plate girder with a light system of lateral bracing.

Swing Span: The Waihou River being navigable the provision of an opening span was necessary. Navigation requirements called for an opening 50ft. wide in the clear.

For a clear width of 50ft. between fenders, the most suitable type of opening was found to be a swinging span. Had the opening been shorter, say 40ft., a lifting leaf type of span could with advantage have been used, being among other things handier and quicker in operation. However, for the wider opening a swing was adopted, and a plate girder span of total length of 140ft. mounted on a central pivot pier was necessary. Thus the total length of bridge was 1,520ft. made up of 23 60ft. plate girder spans and a central swing span of 140ft. Entirely new roading was required for the approaches to the new bridge,—these had to be formed and metalled, land taken and fenced, for a distance of 120 chains across low-lying, swampy ground. After the plans took somewhat definite shape a provisional estimate for the cost of the bridge was made, this being, bridge £46,300, approaches £6,000, total £52,300. Later on, after completion of survey, cost of approaches were estimated at £7,700, so that the total estimate was £54,000, and when plans were finished it was considered structure with its approaches should be completed for that sum.

STRUCTURE.

Commenting now on the plans of the structure it is seen that each pier consists of two groups of piles, the heads of each group enclosed in a cylindrical casing, the whole then surmounted by a dwarf reinforced concrete pier, as described later. In the case of piers under the 12ft. roadway spans, each group consists of three piles while in the case of piers for the spans widened to 18 feet for passing purposes each group consists of four piles. Owing to the difficulty of handling and driving concrete piles of the full length required it was decided to use composite piles, these consisting of a timber pile as a leader to which is spliced the concrete pile. The top of this timber pile is dressed to 10in. x 10in. and a 14in. reinforced concrete pile sliced thereto by 4 iron bark splicing pieces 6ft. long by 8in. x 6in. Generally the timber leader was set in place in the pile frame leaders, resting on the river bottom if long enough and the concrete follower was then spliced thereto and then the composite pile driven to the desired bearing. The piles in each group were intended to be driven at the rake of one in 48 for the 3 pile groups and 1 in 60 for the 4 pile groups, the piles to be driven so that the heads came together. Then a reinforced concrete sleeve or cylindrical casing 5ft. outside diameter with walls 4½in. thick was slipped over the group, the casing being made of such a depth that it would enter 2ft. into the ground and have its top above low-water mark. This cylinder was then to be filled with concrete and a reinforced concrete pier built on the top of each pair.

Pile Driving: The building of a temporary bridge as a staging for construction purposes would have been an expensive matter, considering the great length of the crossing, so the method adopted for pile driving and pier construction was chosen with the view of minimising such temporary construction. A staging of temporary piles was carried forward for such distance as to cover several spans, the platform being at about permanent deck level. From this staging pile driving machines were operated, four temporary piles being driven at the same time at each of the pair of pier cylinders, the necessary false work for building the pier being erected on these. These temporary piles and staging were drawn and continuously moved forward, and thus used several times over. This method of driving from staging was intended to obviate the use of a floating pile driving

plant. The estuary at the site is wide, and is exposed to wind and wave, and tidal currents are strong both on ebb and flow. The driving from staging necessitated the use of a long timber dolly as piles had to be driven to a level 12 feet or more below staging. It was not an easy matter to maintain control of the pile or keep it in position as it was driven away below the derrick. Half-way through the job, the officers in control decided to alter procedure. The pile driving plant was shipped on to a large pontoon and the driving was completed from the floating plant. The change was successful, no particular difficulty was experienced in handling the floating plant in the strong tides that prevailed, and driving was simplified and expedited. It is noted that the officer in charge of the work was not able to detect any difference in the driving itself due to the use of a short dolly from floating plant compared with the driving with long dolly used on the staging.

The test pile driving has already been referred to, and as a comparison of the methods used and results obtained from the test pile driving and the permanent pile driving is of considerable interest, that portion of the work will be described at some length.

Figures 5 and 6 show graphs of the safe bearing obtained with the test piles computed by the "Engineering News" formula. The sets were carefully observed at every five blows by reading with dumpy level. The unusual softness of the first thirty feet penetrated is very apparent. With some of the test piles the safe bearing was almost wholly obtained in the last few feet of driving, see numbers 3 to 6 in particular. These piles obviously encountered harder or more compact beds, very likely sand or gravel beds, but the very irregular graphs in figure 5 indicate also considerable variation in these more compact beds. On the western half of the bridge site, more uniform conditions of driving were met with, and for test piles 7 to 12 the graph of safe bearing approximates to a straight line over the last 40ft. of penetration.

Clearly these last half-dozen test piles derived their bearing power almost wholly by skin friction, and converting their safe bearing values to supporting power per sq. foot of pile surface, a figure of 1½ cwt. per sq. foot is obtained. It should be noted that in driving test piles 4 and 5 the drop was increased from 6 feet to 10 feet and this rather less efficient driving has no doubt given a somewhat higher safe bearing value than if driving had proceeded to the same depth of penetration with the 6ft. drop, though not to the extent to seriously affect the results. The writer considers that test piles, 3 to 6, derive a large proportion of their bearing power by support received at the point of the pile on the harder beds rather than wholly by skin friction as in the case of the other test piles.

The permanent piles were 14in. octagonal reinforced concrete piles spliced to timber leaders. The timber leaders were of kahikatea, except that leaders in piles in piers 2 to 8 inclusive were turpentine. All leaders were shod with 40lb. pile shoes.

Driving of the permanent piles was carried out either with model 9B2 McKiernan Terry double acting steam pile hammer or else with drop monkey 3.2 tons in weight. Data in respect to this model 9B2 is as follows:

Net weight of hammer	6760 lbs.
Weight of piston or ram (moving part)	1500 lbs.
Piston stroke	16 inches
Area of piston striking end	$56\frac{3}{4}$ sq. inches
Blows per minute at 100 lbs. steam boiler pressure	140

The makers of the steam hammer recommend the following formula for the safe bearing:

$$\text{Safe bearing} \;..\;..\; = P = \frac{2\ E}{S + 0.10}$$

When E equals foot lbs. of energy per blow
= 8200 for 140 strokes per minute.
= 7000 for 130 strokes per minute.
= 5940 for 120 strokes per minute.
S = Set in inches per blow.

The number of strokes per minute decreases as the steam pressure at the hammer falls.

Regarding the steam hammer, "Engineering" for 29th May, 1925, page 657, contains a discussion on the kinetic energy of the steam hammer of which the substance is given here:

A = area of piston driving end, and M = steam pressure. The limit of A x M equals weight of the casing of hammer which can preserve contact, or remain resting on pile head during driving.

Therefore value of K. E. becomes stroke x (weight of ram + casing).

The equivalent height of free fall of ram to give the same energy per blow would be $\frac{\text{stroke}\ (W + A\ M)}{W}$, where W equals weight of ram.

This generally amounts to six feet. So applying this to 9 B.2 hammer, K.E. per blow would be 1500 × 6 = 9000 foot lbs. This corresponds fairly well with figure of E = 8200 for 140 strokes per minute as given by makers of hammer.

In "Engineering" for 13th August, 1926, on page 190, is given the "Nichol's" formula for double acting steam hammers.

$$\text{Safe bearing} = \frac{2\ h\ (w + A\ p)}{S + c}$$

where, h = stroke, w = weight of ram
A = piston area, striking end
p = average steam pressure on piston
S = set in inches
and c a coefficient; taken as 0.3 checked by tests with drop monkey.

It is probable, considering the limitations of any formula for this type of hammer that the numerator in the Nichol's formula does not give very different results from the formula using values for "E" given by the makers, where "E" varies according to the number of blows per minute the hammer gives, this number varying as the steam pressure. The use of 0.3 in Nichol's formula for the coefficient in the denominator as checked by tests with drop monkey is worth noting.

It at once became apparent as pile driving proceeded with the steam hammer that the formula $P = \frac{2\ E}{S + 0.1}$, gave excessive values for safe bearing, for instance at pier 24 with final set averaging 115 blows to the inch, that formula gave safe bearing per pile of 57 tons in soft ground at a depth where test pile by "Engineering News" formula would have given a safe bearing of about 16 tons. A more generally used formula is $P = \frac{2\ E}{S + 0.3}$ and as this latter gave more consistent results which fairly well agreed with bearings obtained by tests with drop monkey that latter formula will be used in analyzing bearing values of steam hammer driven piles.

The design of the piers called for a safe bearing of 22 tons per pile by "Engineering News" formula. For piers 25 and 24, piles were driven by the steam hammer close to the limit of its driving capacity, when a set of 1 inch was obtained with over 100 blows. The modified formula $P = \frac{2\ E}{S + 0.3}$ gave a safe bearing of 18 tons per pile. As this was below design requirements, and as the formula obviously had serious limitations, certain piles after being driven with steam hammer were tested with the 3.2 ton drop monkey. Generally the transfer from steam hammer to drop monkey occupied about 20 minutes, and driving with drop monkey continued long enough to make sure the short delay in driving did not influence the test set. The piles as driven would average about 3 tons in weight. The following table compares results by 3.2 ton drop monkey and by steam hammer.

Pier.	Pile.	Depth in ground. feet.	By drop monkey 2 W H S + 1 tons.	By steam hammer. 2 E S + 0.3 tons.	Remarks.
24	6	43		17	Set 100 blows to an inch at rate 120 blows per minute.
24	1	43		17	Ditto.
24	3	43	18.6		Set 1.4 inch with 6′ drop.
23	3	50.3	16.0		This pile driven for the last 9ft. by drop monkey fall 6ft., with uniform set of 1½″ per blow.
23	4	45.5		20	Set 100 blows to an inch, at rate 130 blows per minute.
23	7	46.5		20	Set 130 blows to an inch, at rate 130 blows per minute.
23	2	45.0	22		This pile had set 1½″ per blow for last 12 feet then quickly tightened to set of ¾″.
22	5	63.5		19.3	Set 40 blows to 3″ at rate 140 blows per minute.

Pier.	Pile.	Depth in ground. Feet.	By drop monkey 2 W H S + 1 tons.	By steam hammer. 2 E S + 0.3 tons.	Remarks.
22	8	61.1	25	19.4	Though pile tested with drop monkey gave bearing of 25 tons pile was driven with steam hammer for further 4ft., and gave bearing of 18 tons.
16	3	40.0 49.0	25.0 29.5	20.8	Driven to refusal with steam hammer at depth 40ft. Then driven 9 ft. with drop monkey.
16	6	38.8 40.8	21.0 25.6	18.2	See footnote.* Final set with drop monkey was ½″ per 6′ blow.
12	2	55.3	30.0	22.2	

Pile driving commenced at the Eastern end of bridge at Abutment 26, and for the first few piers the driving was stopped at an earlier stage than later experience showed as desirable, probably under the influence of formula, already referred to, recommended by makers of steam hammer. In several of the piers, the driving had been discontinued with the pile moving at the rate of an inch of set to 30 to 50 blows of the steam hammer, piles having penetrated varying distances from 54 feet to 38 feet in the ground (at pier 22 where piles penetrated most freely penetration was from 61 feet to 67 feet in the ground, one pile when driving ceased, moving 3in. to 40 blows.

Instructions were then given that driving was to be continued to the practical limit of the steam hammer capacity, and subsequent work was all to that standard. In view of the nature of the driving that had to that stage been experienced it was decided to subject one pile to an actual test by loading it. Accordingly a pile exactly similar to permanent piles was driven just

*All piles in pier 16 pulled up very quickly. Piles sunk 32 feet in ground under weight of pile and steam hammer, total of about 7 tons, and piles were driven for about 7 feet only.

clear of the bridge between piers 15 and 16. Details of pile are as follows:

Wooden leader, 25ft. 6in. long; R.C. follower, 35ft. 6in. long. Total, 61 feet.

Ground level, 71.50ft.; level of head, 96.50 feet.

Level of point, 35.50 feet.

(The pile was thus 36.0 feet in the mud, of which distance, 25ft. 6in. was timber leader, and 10ft. 6in. concrete pile).

Mean high tide, 97.2ft.: Mean low tide, 88.0ft.

Pile sank 27.50ft. under dead weight of pile and hammer with point at 44.0ft.

With point at Reduced Level, 37.50ft. 5 blows to an inch set gave 14.4 tons bearing.

At Reduced Level, 36.50ft. 13 blows to an inch set gave 19.1 tons bearing.

At Reduced Level 35.58ft. 170 blows to an inch set gave 23.5 tons bearing.

All at 139 blows per minute, with the steam hammer.

The safe bearing was not tested with drop monkey, but it would probably have given a figure somewhat greater than 23.5 tons.

The method of applying the load was as shown in figure 7. It will be noted that the load of rails was so applied and balanced that the test pile carried 53/54 of the total load applied to the platform.

The total load on test pile was 63.16 tons, at which loading there was a gross settlement of 11/16 inch.

The accompanying graph (figure 8) shows the history of the loading and settlement and also the recovery effected when the load was reduced.

It would appear that the pile had an initial resistance to settlement—or inertia—of 21 tons. It then settled 3/16 inch fairly suddenly and from that point settled fairly evenly with the application of the load to a maximum of half inch at full load. After this, the pile was subjected to very heavy weather and rough seas which swayed the load about, and subjected the pile to a torsional strain. The pile settled 3/16 inch under this treatment, the load was then reduced to 33½ tons, with a recovery of ⅛ inch and then again increased to 44½ tons, without settlement. This load was maintained over three months without further settlement. Unfortunately after this interval the opportunity of measuring the final recovery on removal of whole load was lost as the original base from which the levels were taken had been concreted up. From the graph, however, it is feasible

that an elastic recovery of all but 3/16 inch was effected and further that, but for the final settlement due to bad weather, the whole settlement was elastic.

Having regard to the very considerable depth of soft mud in which piles were founded and to the possibility that a group of three piles in such a bottom so closely driven together would not necessarily give a bearing three times the value of bearing for a single pile, and to the fact that piles were driven with steam hammer for which results by formula cannot be accepted without confirmation, an actual load test on completed pier was made.

Pier 21, built on to two groups of three piles each, was selected. These piles average 50ft. in the ground and finished with an average set of one inch to 40 blows. The finishing driving was about the lightest in the whole structure and pier 21 was chosen for that reason. The pier was subjected to a static load equal to its total dead and its live design load. The pier carried this load for at least 6 weeks without movement.

The results of test loading of pile and pier were considered satisfactory. Nevertheless for all subsequent driving including piers 16 to 1, the requirement in respect to the set was stiffened up and piles were driven to a set of from one inch per 100 blows to one inch per 250 blows where steam was used.

By the time the piles of pier 7 were reached the steam hammer began to prove inadequate to keep piles moving through the big depth of uniform ground to a point at which piles would give a satisfactory set under the drop monkey. Accordingly piles of piers 7 to 2 were driven with the 3.2 ton monkey and finished up with sets round about ½ inch or less per blow.

By an examination of the table given in preceding pages comparing steam hammer and drop monkey tests, and comparing results for pairs of tests at similar depths, it is seen these tests give an average safe bearing of 22.5 tons for drop monkey by "Engineering News" formula, and 19.6 tons by steam hammer formula, the latter figure being 87 per cent. of the former. It is also apparent that in the easier driving showing in the first part of the table the results compare fairly well, but in the stiffer driving when piles in piers 22, 16 and 12 are compared the difference becomes wider.

The formula $\dfrac{2\ E}{S + 0.3}$ would be made to accord with results given by drop monkey at Kopu Bridge better by reducing the constant in the denominator, making it read $\dfrac{2\ E}{S + 0.26}$

It is obvious that in the formula $\frac{2\ E}{S + 0.3}$ assuming steam pressure constant so that E is constant, the bearing obtained is almost wholly governed by the constant 0.3 in the denominator, for the set per inch is a very small fraction of 0.3. For instance, in the driving at the Kopu Bridge the set varied from 0.07 (very little of this) to less than 0.005. Therefore as a guide its results cannot be accepted for safe bearing unless results are closely controlled and checked by determinations by drop monkey.

It is also apparent that the steam hammer of the size used in the class of foundation as experienced at Kopu reached the extreme upper limit of its capacity at a safe bearing of barely 25 tons determined by drop monkey. Several of the tests showed that when the steam hammer had almost reached refusal, the drop monkey was able to keep piles moving freely (see piles in pier 23). Nevertheless, under suitable conditions this type of hammer has considerable penetrative powers, take pier 22 for instance, already quoted. The practically continuous movement of the pile under the rapid blows probably develops lubricated friction. Some time after driving and when ground has had time to settle and become compacted, friction might revert to static friction.

Figure 9 is a copy of a plan on which a comparison is made of the results of test pile driving and of the bearings obtained with the permanent piles. It will be noted that depths to which test piles and corresponding permanent piles were driven did not on the whole vary greatly.

Safe bearings obtained by steam hammer formula $P = \frac{2\ E}{S + 0.3}$ are marked in a different way to those determined by drop hammer. The writer has already commented that $P = \frac{2\ E}{S + 0.3}$ gives results averaging 87% of corresponding results by drop monkey tests, but correcting the former to bring them into accord with the latter, we obtain the rather surprising result that the bearing given by permanent piles is practically double of that given by the test piles. This striking difference is especially clear when test piles 10, 11 and 12 are compared with permanent piles in piers 7 to 2 inclusive, all these piles being driven with the drop monkey. Now the test piles were driven without splicing pieces, the follower being held to the leader by a simple corset. On the other hand, the splicing pieces of the permanent piles increased the surface area per pile by about 12 square feet, and from results at this job, skin friction on this

extra area would probably give additional safe bearing of from 20cwt. to 30cwt. As far as the writer can ascertain the diameter of permanent piles would on an average exceed that of test piles by 10% to 20%, the area subject to skin friction being increased in like ratio. Probably the more important factor in giving increased bearing for permanent piles was the greater coefficient of roughness of the latter. Test piles were stripped clean of bark, whereas leaders of permanent piles were driven with bark on. The concrete pile would also have a greater coefficient of roughness than test piles, but as the concrete portion of pile only penetrated the softest part of bottom, its effect would be small.

It is regretted that a closer investigation of this circumstance cannot be made. Nevertheless the important fact is that the big difference in bearing was there, proved by careful observations, and the circumstance is worth bearing in mind in considering pile foundations in the light of test pile records where bearing is mainly developed by skin friction.

The writer does not intend to comment on the general use of the "Engineering News" formula instead of a formula of a type such as "Weisbach's." This subject was very ably treated by Mr. Ashley Hunter in Proc. N.Z. Soc. C.E., Vol. 13. This remark may be permitted, that the use of the "Engineering News" formula in the Public Works Department is generally under conditions where the weight of monkey bears a proper relation to the weight of pile, and where the set comes within certain mean limits. Under these conditions its use has given satisfactory results.

General Construction.

Earlier in this paper reference is made to the provision in the drawings for a rake on the piles of 1 in 48 for the 3 pile clusters and 1 in 60 for the 4 pile clusters. Difficulty was experienced in driving to these rakes mainly because of the very limited amount of room in the cylinder casing to make sure of bringing the pile heads together and yet maintain the piles to the specified rake. Generally that requirement was not very well complied with. As the bridge approached completion it was found that instead of the girders moving on the bed plates at expansion ends the piers tended to move in a longitudinal direction under expansion and contraction of the deck; this movement being cumulative in one direction. This in the main was due to the exceedingly soft ground in which the piles were driven. To overcome this a mattress of stone was placed upon the river bed around certain of the piers and it has been found that this has afforded the necessary resistance to movement by the pile piers.

The limits of this paper do not allow much space for comment on the routine construction. Generally the work was straightforward, and was carried out to programme without difficulty. Special attention was given to the upper surface of the top chord of the steel girders so as to ensure a perfectly clean surface for the concrete to bond on to. To provide additional bond and to assist in transferring shear between steel girder and concrete deck angle iron cleats 3in. x 2in. x ½in. x 8¼in. long were rivetted to the top flange, being spaced at intervals varying from 2ft. to 3ft. centres, throughout the whole of the steel joists.

Great care was taken to have the boxing for the concrete deck thoroughly well fitted to the steel flange to prevent any loss of cement mortar so that the concrete over the steel and indeed over the whole deck would be of maximum density and without open spaces so as to give full protection to the steel. It is noted that the success of this design, made as it was for economy and efficiency, is wrapped up in proper construction to protect the steel and in the subsequent care of the steel girders by adequate painting maintenance. On this work, this requirement has received most careful attention.

The fabrication of the steel girders was carried out in the Department workshops at Tauranga. The girders ready for erection were transported by scow round Cape Colville direct to the site and at favourable stages of tide lifted straight on to the piers by the scow's winches. This proved very convenient and economical.

As the approaches to the bridge were across low alluvial and mangrove flats, the earthwork was considerable. The borrow pit method of construction would not have been suitable, so advantage was taken of the existence of the Department's 10 inch suction dredge which had carried out a big programme of work up the river. Sand, mud and shell were pumped ashore to dumps on either bank, and later run out to filling, making an excellent road bank.

The whole of the concrete work, including cylinder and pile fabrication, as well as concrete work on the bridge itself, and all pile driving, both temporary and permanent, and erection of falsework was carried out on the co-operative system. This

WAIHOU RIVER BRIDGE AT KOPU

SUMMARY OF COSTS.

(Including supervision and administration £2,300.)

	£	s.	d.	Quantity	Unit cost	Cost per ft. bridge.
Temporary Staging	£ 5,587	3	9			£3 13 2¼
Splicing & Driving Piles	11,036	16	5	12,106½ ft.	Cost per ft. driven £0.18.3 p. ft.	7 4 10
Cylinders (132 c.y. reinforced concrete 300 c.y. mass concrete)	4,162	17	5	46	" " Cylinder 90.9.11 each	2 14 7½
Decking*	5,412	4	7	2,109 sq. yds.	" " sq. yard 2.11.4 p. sq.yd.	3 11 0
Handrails	1,800	4	7	1,524 lin. ft.	" " lin. ft. 1.3.7½ p. l. ft.	1 3 7½
Abutments & Piers	3,887	19	6	261 c. yds.	" " c. yd. in place 14.17.11 p. c. yd.	2 11 0
Fairway	3,134	15	4	576 lin. ft.	" " lin. ft. 5.8.10 p. lin.ft.	2 1 1½
Girders	13,050	9	2	392½ tons	" " ton in place 33.5.0 p. ton	8 11 3¼
Swing Span (excluding girders)	4,289	15	0		Lump sum in place	2 16 3
Test Pier	319	16	1			4 2¼
Approaches	7,706	18	8	83 chains	Cost per chain 92.17.0 per ch.	
	£60,389	0	6			£34 11 1¾

Cost per ft. of Bridge, including Approaches	£34 11 1¾
Cost per ft. of Bridge, excluding Approaches	30 2 11¼
Cost per sq. yd. of deck, including Approaches	24 19 7
Cost per sq. yd. of deck, excluding Approaches	21 15 10
Cost per sq. ft. of deck, excluding Approaches	2 8 5

*Decking consists of 7½" of reinforced Portland cement concrete and 2" of bituminous concrete. The quantity of concrete in the kerbs would be equivalent to an additional 2" in depth over the deck.

method was successful, though it is not very flexible or suitable when it is necessary to push a job along.

Space does not permit of description of the swing span and of the operating mechanism. The design was very successful. The parts went together perfectly and the mechanism operates like clock-work.

Careful cost accounts were kept of the work, and total expenditure amounted to £60,389. A summary of costs subdivided into main items is appended.

The design of the bridge was due to Mr. J. E. L. Cull, the then Designing Engineer of the Department. The immediate construction was carried out under the control of Mr. O. G. Thornton, Resident Engineer, and Mr. A. P. Grant, Assistant Engineer. It is to these officers that the successful carrying out of the work and the careful collection of a very great deal of information appearing in this paper is due.

Mr. H. F. Toogood said that the most valuable portion of the paper was that relating to the pile driving, and the loading tests. Nothing was of greater interest to the bridge builder than a further insight into the loading piles may be expected to carry in soft foundations. He regretted that Mr. Baker had not given a few more deductions from the ascertained facts. Only the *Engineering News* formula was referred to in determining the bearing power of piles, and no reference had been made to its very obvious failure to give results even approximately in accordance with those determined by actual loading. In the pile loaded for testing purposes that formula indicated that the safe load would be some 23 tons, but when loaded with 21 tons the pile started its movement. Concerning the initial movement up to ¼″ he would refer to this later, but it was to be noted that the pile continued to move until it had settled 11/16″, and then sustained a load of 60 tons, or less than three times the calculated safe load, and not four to six which the *Engineering News* formula could be expected to give under certain proportions of weight of pile to ram and drop used. This settlement was far too great for a reinforced concrete bridge with continuous girders, and if the indicated safe load had been adopted, would have led to serious results. It was to be noted that the pile tested recovered settlement amounting to ⅛″ when the load was reduced to 33 tons, due to the resilience of the pile, and possibly also to the resilience of the ground. With regard to the initial settlement, he had a graph taken of the actual behaviour of a pile from the time of impact to the time of quiescence, which

might help to explain at least a portion of the first stages of settlement as observed.* The graph was taken at the Avon River Bridge by holding a pencil rigidly on a cleat bolted to the pile, and so that its point touched a piece of paper held on a sliding board attached to the frame, which, by the way, was independent of the pile or ram, once the ram was tripped. As the ram dropped, the slide with the paper attached was moved uniformly across until the result of the blow had ceased, the pencil being held firmly against the paper. The result was a graph recording the complete movement of the pile from the time of impact. It showed that while the actual set was in the order of ⅝″ the total penetration was 1″. In other words, the head of the pile was driven 1″ and came back ⅜″. There was a characteristic tip on each curve, which amounted to 1/16″, which he believed was the resilience of the concrete pile, and the remainder of the ⅜″ of rebound was the resilience of the ground. The pile, after penetrating 1″, rebounded ⅜″, and with considerable velocity, and at the end of its upward movement must possess considerable inertia and perhaps sufficient to actually make the pile leave a space between the consolidated ground at the toe and the pile shoe, which might account for at least ¼″ of the initial set. If this was so, it indicated further driving of a light nature was required to seat it into the consolidated ground, and perhaps this should be incorporated in future specifications. The graph was taken by Mr. Maurice Wright, A.M.N.Z.Soc.C.E., in order to test the pile driving results of the Avon River Bridge by Hiley's formula, and gave the missing compression factor of the ground required in that formula. At the Avon River Bridge, he had been somewhat concerned as to the carrying capacity of the piles. The foundations were not good, and reinforced concrete piles 14″ octagonal and 33′ 0″ long designed to carry 14 tons each, were required. This was light loading, but he did not feel safe in risking more. Test piles were driven and at the depth to which the 33′ piles were to be driven gave, by the *Engineering News* formula, a safe loading of 9 tons. These piles were old electric line piles, dressed ironbark 24″ circumference at toe and 33″ circumference at butt. By taking the proportional areas exposed to skin friction the 14″ octagonal piles should be safe for 13.8 tons, but believing that the close piling of 3′ 0″ centres would result in consolidating the ground, he allowed 14 tons. For the permanent piles driven the results of Hiley's formula indicated an ultimate load of 100 tons, which with a factor of safety of six was approximately 17 tons safe load. He might add that they did not have the financial resources behind them

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*Fig. 10.

to actually load a pile. If they would accept the *Engineering News* formula as reliable in the circumstances in which it was used, they would note that Mr. Baker had a somewhat similar experience at Waihou. He found that if the area exposed to skin friction were the basis of the calculation, the larger permanent pile gave a proportionately higher bearing power than the smaller test pile indicated. With regard to the use of a dolly, it might be considered surprising that no difference in set was observed at the Waihou Bridge. At the Avon River Bridge the average of many readings showed that with a 3.25 ton ram, a 14″ octagonal reinforced concrete pile 33′ 0″ long, and an ironbark dolly 10′ 0″ long and weighing 8 cwt., it required a 4′ 0″ blow to give the same set as that obtained with a 3′ 0″ blow when only a helmet was used, the set in both cases being in the order of $\frac{3}{4}$″. The influence of a dolly must be in proportion to the other resilient factors, and if its influence was not felt at Waihou it would indicate to him that the resilience without the dolly was very high, and the addition of the dolly effected the whole by a small amount. The history of the loaded pile indicated this also, showing what a large amount of the energy of the ram had been absorbed by resilient factors and how disappointing the *Engineering News* formula was under such circumstances. He thought that Mr. Baker had stopped his reference to pile driving at the most interesting point. When he read the paragraph as follows he hoped he would have continued. Mr. Baker said: "The use of the *Engineering News* formula in the Public Works Department was generally under conditions where the weight of monkey bears a proper relation to the weight of pile, and where the set comes within certain mean limits. Under these conditions its use has given satisfactory results." This was quite so; the *Engineering News* formula, while quite safe if properly used, became an absurdity under certain conditions, and what would be of real value to them would be a statement as to the safe limits under which it might be used. This, he thought, was the logical conclusion, and he hoped time would be found in the written reply to supply at least the writer's deductions as to these limits. He further hoped Mr. Baker would attempt to apply Hiley's formula to the results obtained at Waihou Bridge.

Mr. Mead thanked Mr. Baker for the great amount of useful information given about the construction of this bridge, especially about the pile-driving. It would be of great value where piles had to be driven through deep alluvial material. If some did not agree with the formula used, the information was given in

the paper to enable them to test it. He wished to ask Mr. Baker a question arising out of Diagram 9. In piers 16 and 22, the pile as finally driven went considerably below the expected figure from the test piles. Was it necessary to add to the length of the concrete pile to get it down; how, if so, was this done, and did it hold up the job at all?

Mr. W. A. Gray said that the following figures about the Whau Bridge—inspected by members the previous day—might be of interest. The existing wooden one-way bridge was 175 ft. long. The new bridge was to be 83 ft. long, with scoria-filled approaches running through the abutment columns, and protected with blue stone pitching. The width was to be 46 ft. between parapets, 32 ft. between kerbs, and concrete paving was to be laid on the filling a year after the bridge had been opened to traffic, with temporary bitumen emulsion paving on green filling. The cost of the bridge proper on the contractor's schedule was approximately 17/6d. per square foot, and the contract price divided by the figure 175 by 46 gave approximately 15/6d. per square foot. Some of the schedule prices might be noted: reinforced concrete, 3 : 1½ : 1 (light reinforcing), £4/14/6 per cubic yard; reinforced concrete, 4 : 2 : 1 (heavy reinforcing), £4/12/6. Scoria fill brought five miles with no return load and including all charges, 3/- per cubic yard; blue stone pitching (smallest dimension 12 inches), grounted 8/- per square yard. The concrete price was 25 per cent. below the estimate, which was prepared a few months before the tenders were received.

Mr. H. W. Beasley considered Mr. Baker's paper one of the most valuable that had been brought before the Society, particularly with regard to the experience of pile-driving. In the Wanganui district he had a somewhat similar problem in connection with the Whenuakura bridge. Like the conditions on the Waihou, a very fine alluvial deposit had to be contended with, borings to 150 ft. showed nothing at all solid, so it was decided to drive two test piles, one on the south bank of the river and the other on the north. The first was an ironbark pile driven 57 ft. below ground level, and dollied for another 7 or 8 ft. The diameter of the pile was 18″ at the butt, tapering to 10″ at the point. That pile was tested with 92 tons of rails on it, and there was no settlement. The one on the north bank was driven 53 ft., and was likewise tested with 90 tons of old rails, and in neither case was there any settlement. In the case of the pile on the south bank they got by calculation a bearing power of 79 tons and on the one on the north bank 53 tons. That was the safe load. The formula used was the Saunders (N.Z. Railways), in which the bearing power P (in tons) $= \dfrac{W\ H}{13\ D}$ where W = weight of monkey in cwts., H = drop in feet, and D = the average penetration in inches per blow for the last foot driven. The monkey used weighed 42 cwt., and the pile two tons. In the last one there was a drop of 12 ft. with an average set of $\frac{1}{2}''$. The test established the fact that each pile could support from 90 to 100 tons, and a working load of 30 tons was adopted. The design provided for two cylinders with 14 piles in each cylinder, 40 ft. long, driven into the bed of the river. He had no doubt that the piles would stand up to the required loads.

Mr. F. J. Jones congratulated Mr. Baker on his excellent paper, particularly as regards the pile-driving. He had known many thousands of piles driven in all sorts of ground, and he did not know that any very particular difficulty arose. The practice of the Railways Department was that if they got into soft ground, such as tidal mud, and found they could not get a bearing, they would put cradles on to the piles, not very far from the bottom of the pile, about 8 or 10 ft. from the bottom, just heavy pieces of ironbark, bolted in the two directions. The cradle pieces were from 2′ 6″ to 3′ long. These had carried them over their difficulties many times and the piles so equipped had stood for many years carrying heavy locomotives. As a rule a bridge pile carried as heavy a weight as any type of pile was required to carry.

The President said he would like to ask Mr. Baker one question: there were three piles driven, and the cylinders were dropped over these piles. The cylinders were supposed to enter the mud for a depth of 2 ft. Did the current of the river allow that to be done?

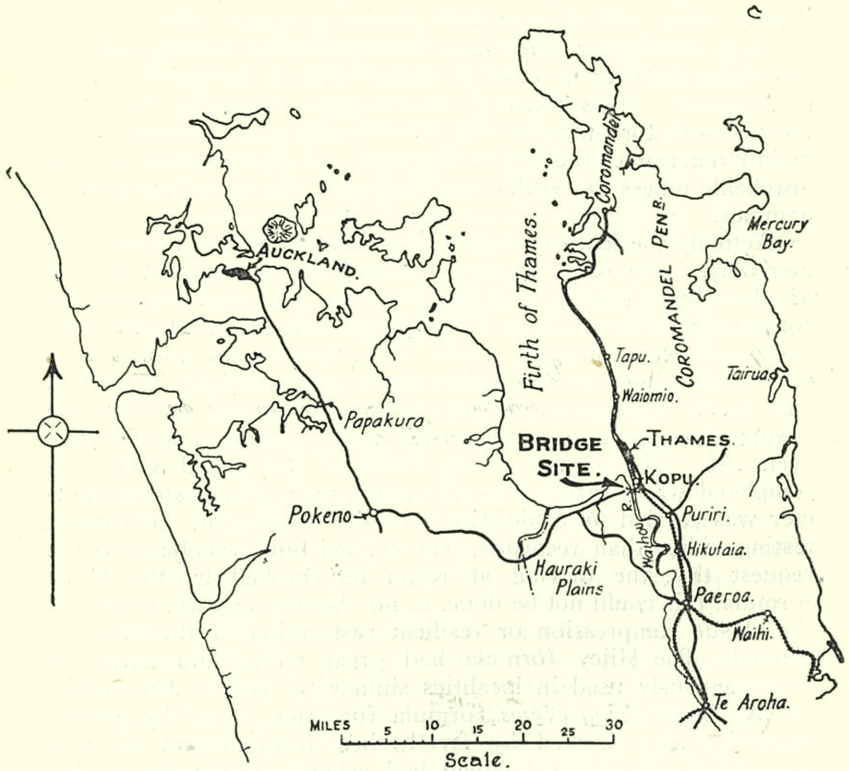
Mr. H. W. Beasley said that in connection with the results they got with the test piles driven at Whenuakura, he had calculated the bearing power by the *Engineering News* formula; on the pile in the north bank the safe load by calculation was 22.4 tons. These piles had been loaded up to 93 tons, and there was no settlement whatever observed. The factor of safety over the calculated safe load was thus four, which went to show that in the conditions met with the *Engineering News* formula gave unduly conservative results.

Mr. A. J. Baker thanked members for their reception of his paper, particularly Mr. Toogood for the very keen interest he had taken in the question of pile-driving. At the start of construction the double acting steam hammer was adopted for the main portion of the work, but in his opinion they would not use

it again for work of that class, because it had, to a considerable extent, proved itself inadequate. Mr. Toogood had suggested that the *Engineering News* formula failed to give results. That would be an incorrect or incomplete deduction. For those piles driven by the standard gravity hammer the values for safe bearing given by the *Engineering News* formula are not open to challenge. The paper explained, however, that pile-driving done with the steam hammer had to be checked and controlled by driving done with the standard gravity hammer. Further, that to apply the *Engineering News* formula to the driving done by the steam hammer a constant of 0.26 had to be applied in the denominator. This constant so outweighs the other factor in the denominator—that is "the set"—that the constant itself mainly determines the results obtained, and the formula becomes unreliable unless its results are checked by the use of the gravity hammer.

Actually the loaded test pile was driven with the steam hammer only. The Kopu Bridge is situated in an exposed portion of the river, and during the test the pile was exposed to considerable wave action. It commenced to settle with 22 tons, and settlement continued until, with 60 tons, it amounted to about half-an-inch. It was clear that a portion of the settlement was due to elastic compression of the ground and that had the unsupported pile not been exposed to wave action it would have been less. Though the testing indicated that the driving then completed was "safe," subsequent driving with the steam hammer was carried to a definitely harder stage, but no further testing, other than recorded, was carried out. Replying to the request that the driving at Kopu be checked by the Hiley formula, this could not be done, as no observations were taken of the elastic compression or resilient restoration of the pile and ground. The Hiley formula had great merit, and might be advantageously used in localities similar to Kopu. The results by the *Engineering News* formula for piles driven by gravity hammer were checked by Eytelwein's formula and were in reasonable accord. Mr. Mead had asked whether the driving of the longer piles in piers 16 and 22 had involved loss of time or a hold-up due to lost time extending piles. There was no lost time; the extra length of pile was obtained by the use of a longer length of timber "leader." The President had asked whether there was any difficulty in getting the concrete cylinder over each group of piles, to enter the mud bottom for a depth of two feet as required by the drawings. The current did not present any difficulty, as in any case the cylinders could be placed in dead water at the turn of the tide. Generally the cylinders were not sunk the full two feet, as the cost of the necessary excavation was not warranted, particularly seeing that the soft mud bottom continued for an indefinite distance below the river bed.

A vote of thanks was accorded to Mr. Baker for his paper.

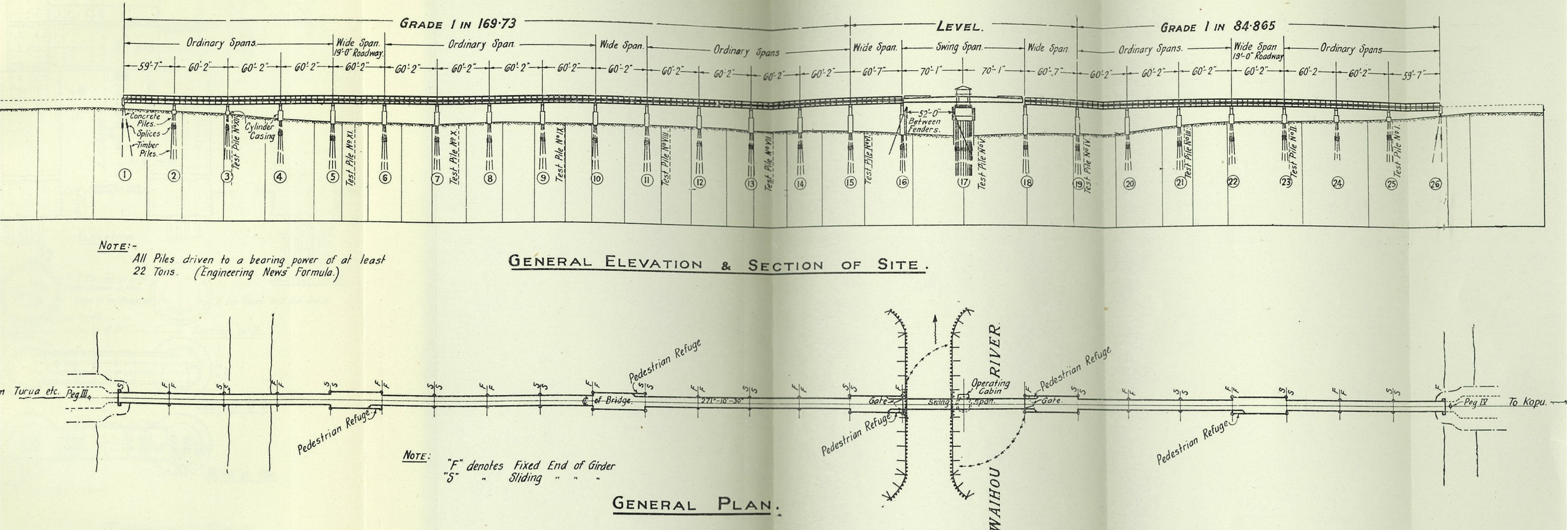


LOCALITY PLAN.

WAIHOU RIVER ROAD BRIDGE:

KOPU.

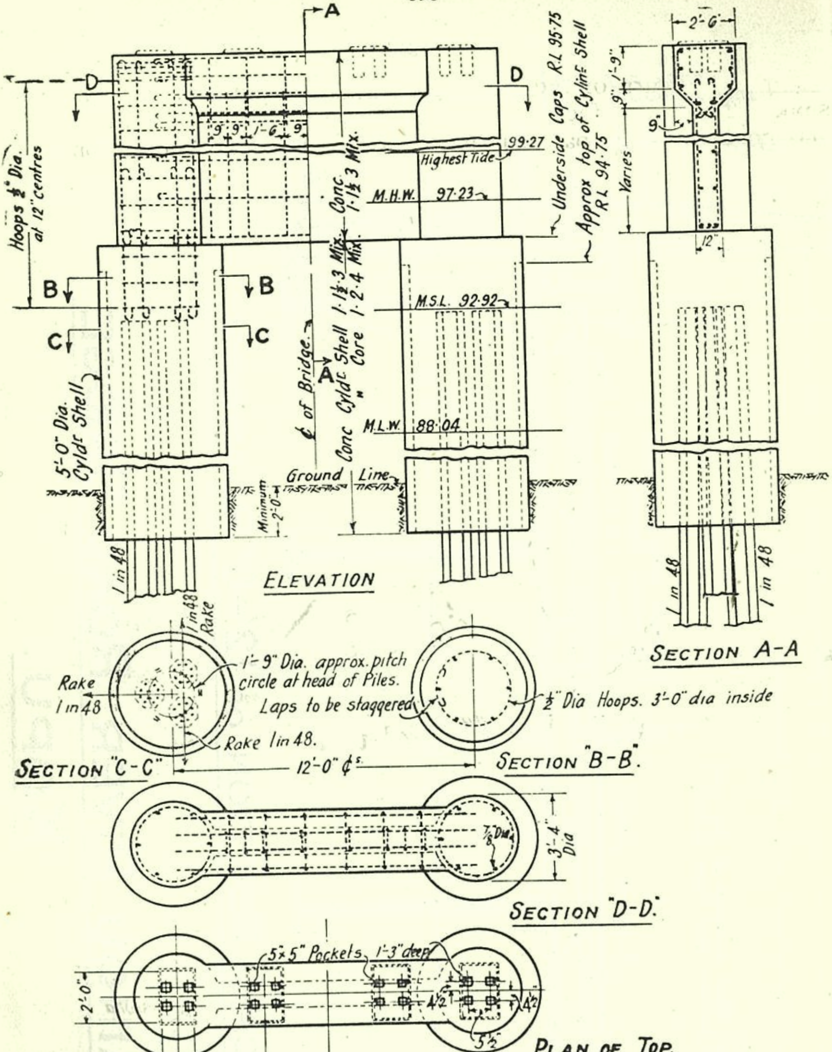
FIG. I.



WAIHOU RIVER ROAD BRIDGE.

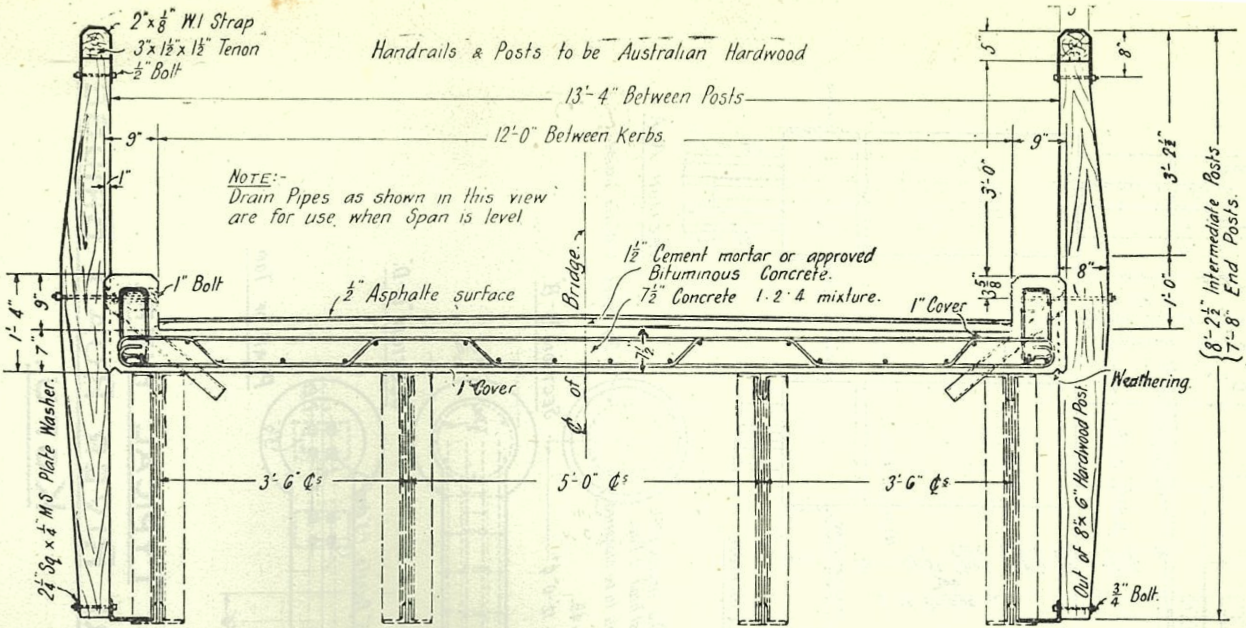
KOPU.

FIG. 2.



TYPICAL PIER.
WAIHOU RIVER ROAD BRIDGE
KOPU.

FIG. 3.

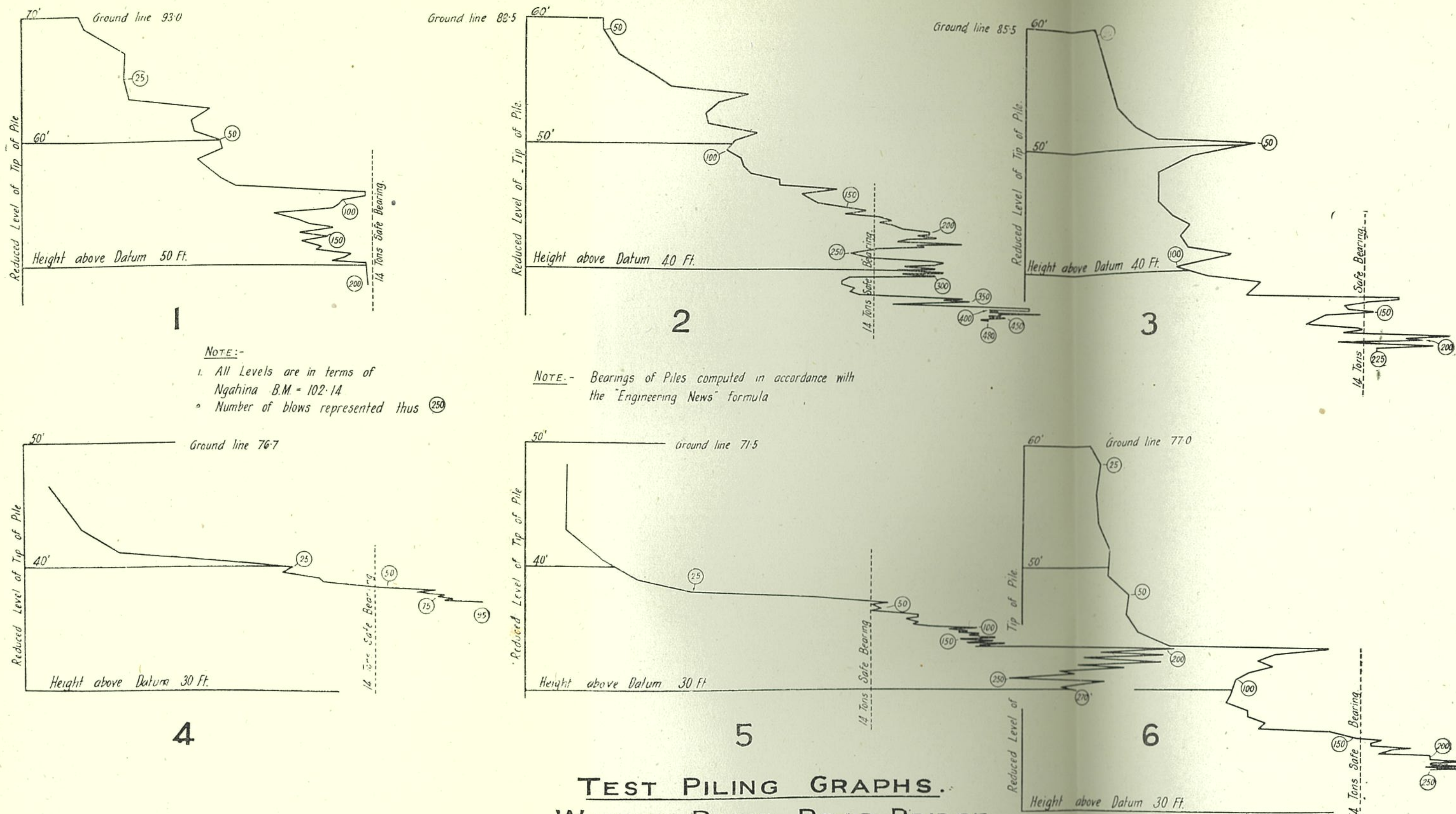


TYPICAL SECTION OF 12'-0" ROADWAY.

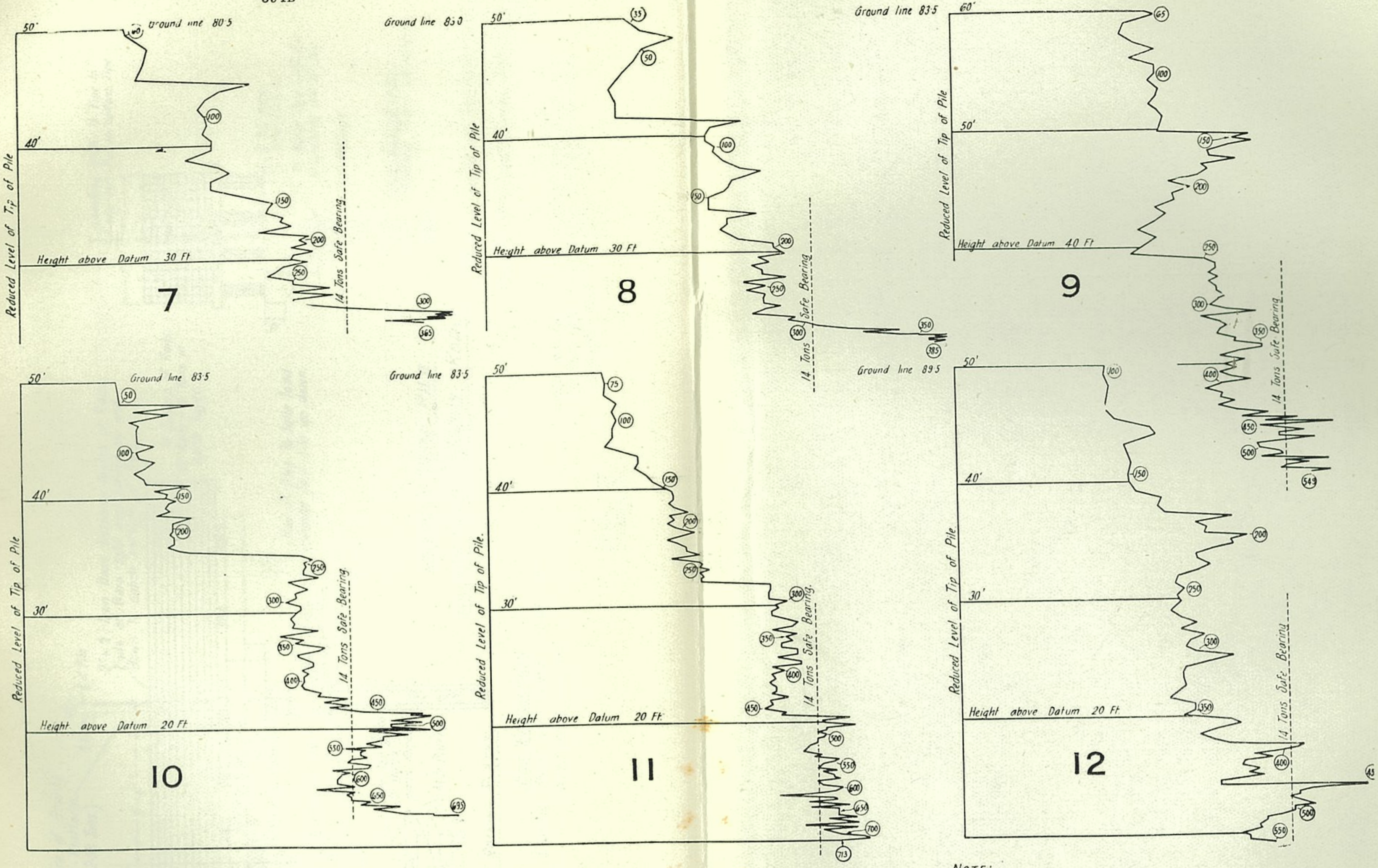
WAIHOU RIVER ROAD BRIDGE.

KOPU.

FIG. 4.



TEST PILING GRAPHS.
WAIHOU RIVER ROAD BRIDGE.
KOPU.
FIG. 5.



NOTE:-
1 All levels are in terms of Ngahina B M = 102·14
2 Numbers of blows represented thus (250)

NOTE:-
Bearings of Piles computed in accordance with the "Engineering News" formula

TEST PILING GRAPHS.
WAIHOU RIVER ROAD BRIDGE.
KOPU.
FIG. 6.

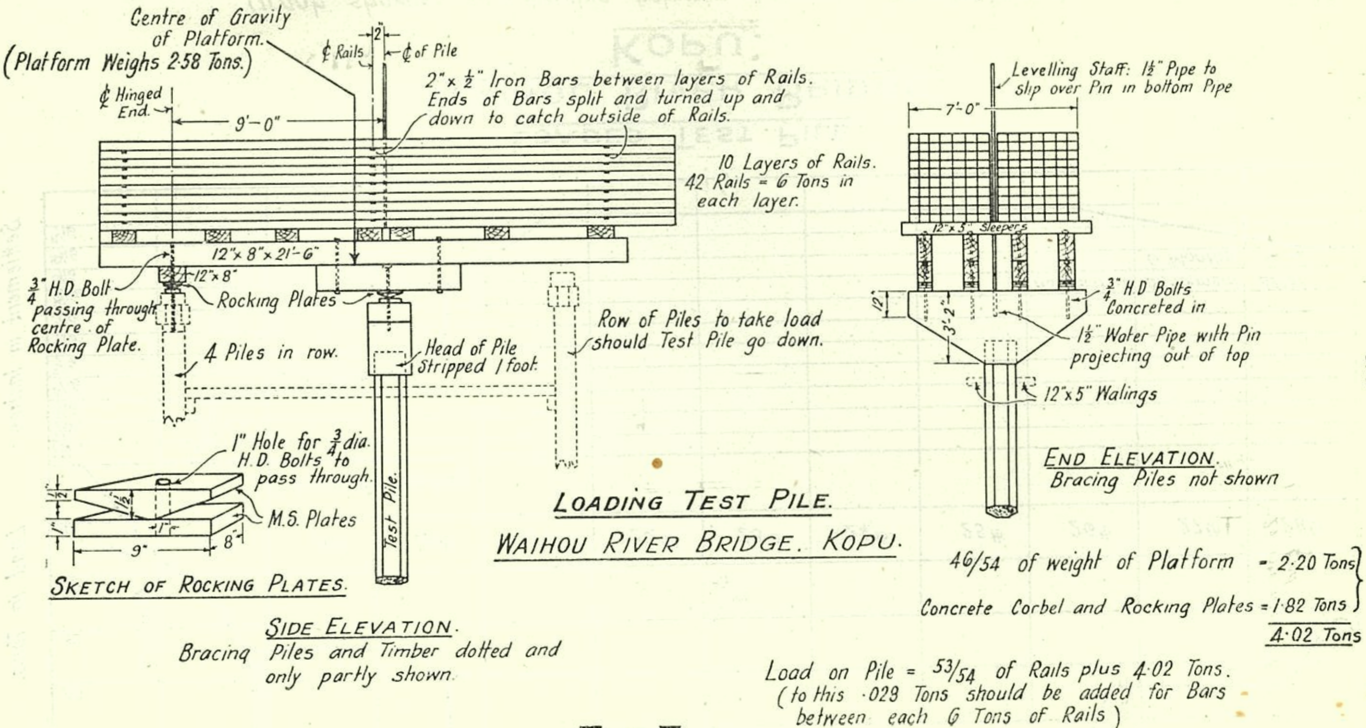
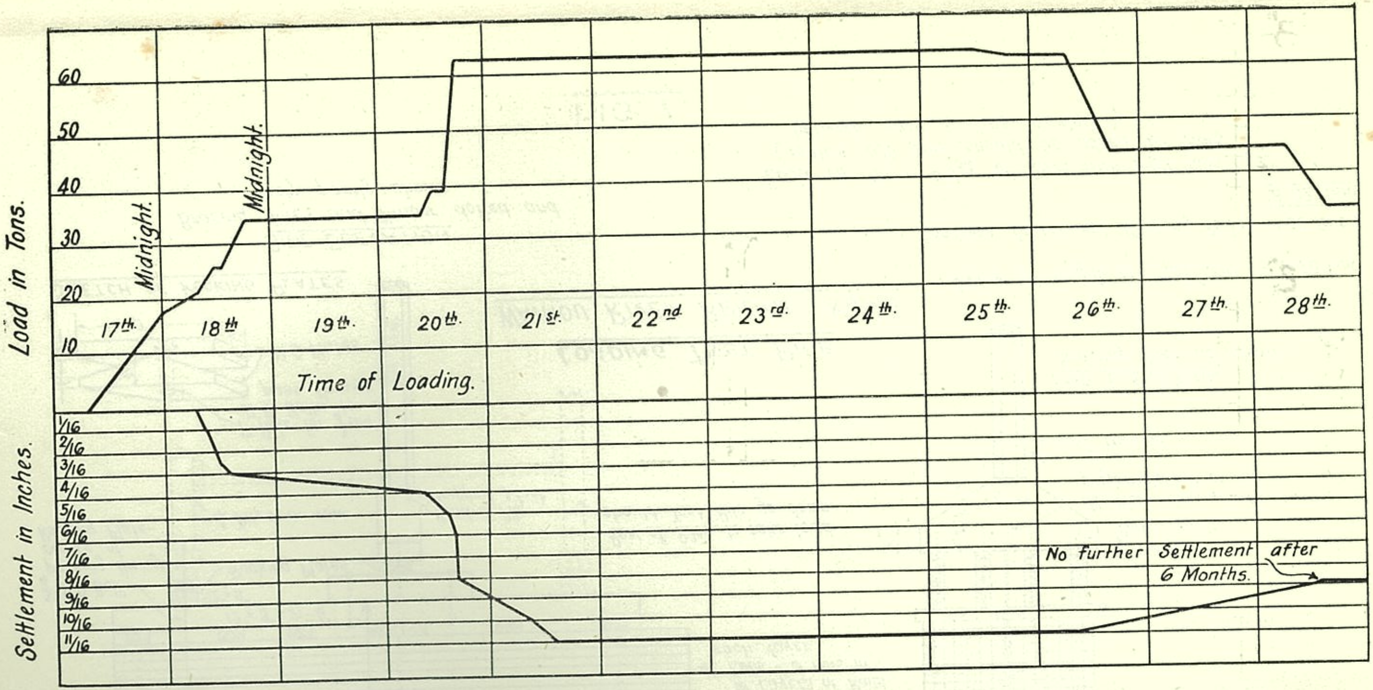

Centre of Gravity of Platform.
(Platform Weighs 2·58 Tons.)
℄ Rails
2"
℄ of Pile
2" x ½" Iron Bars between layers of Rails. Ends of Bars split and turned up and down to catch outside of Rails.
℄ Hinged End.
9'–0"
10 Layers of Rails.
42 Rails = 6 Tons in each layer.
12" x 8" x 21'–6"
12" x 8"
¾" H.D. Bolt passing through centre of Rocking Plate.
Rocking Plates
4 Piles in row.
Head of Pile Stripped 1 foot.
Row of Piles to take load should Test Pile go down.
Test Pile.
1" Hole for ¾ dia. H.D. Bolts to pass through.
1½"
1½"
1"
1"
M.S. Plates
9"
8"
SKETCH OF ROCKING PLATES.
SIDE ELEVATION.
Bracing Piles and Timber dotted and only partly shown.
LOADING TEST PILE.
WAIHOU RIVER BRIDGE. KOPU.
Levelling Staff: 1½" Pipe to slip over Pin in bottom Pipe
7'–0"
12" x 5" Sleepers
12"
3'–2"
¾" H.D Bolts Concreted in
1½" Water Pipe with Pin projecting out of top
12" x 5" Walings
END ELEVATION.
Bracing Piles not shown
46/54 of weight of Platform = 2·20 Tons
Concrete Corbel and Rocking Plates = 1·82 Tons
4·02 Tons
Load on Pile = 53/54 of Rails plus 4·02 Tons.
(to this ·029 Tons should be added for Bars between each 6 Tons of Rails)

FIG. 7.

395



LOADED TEST PILE.

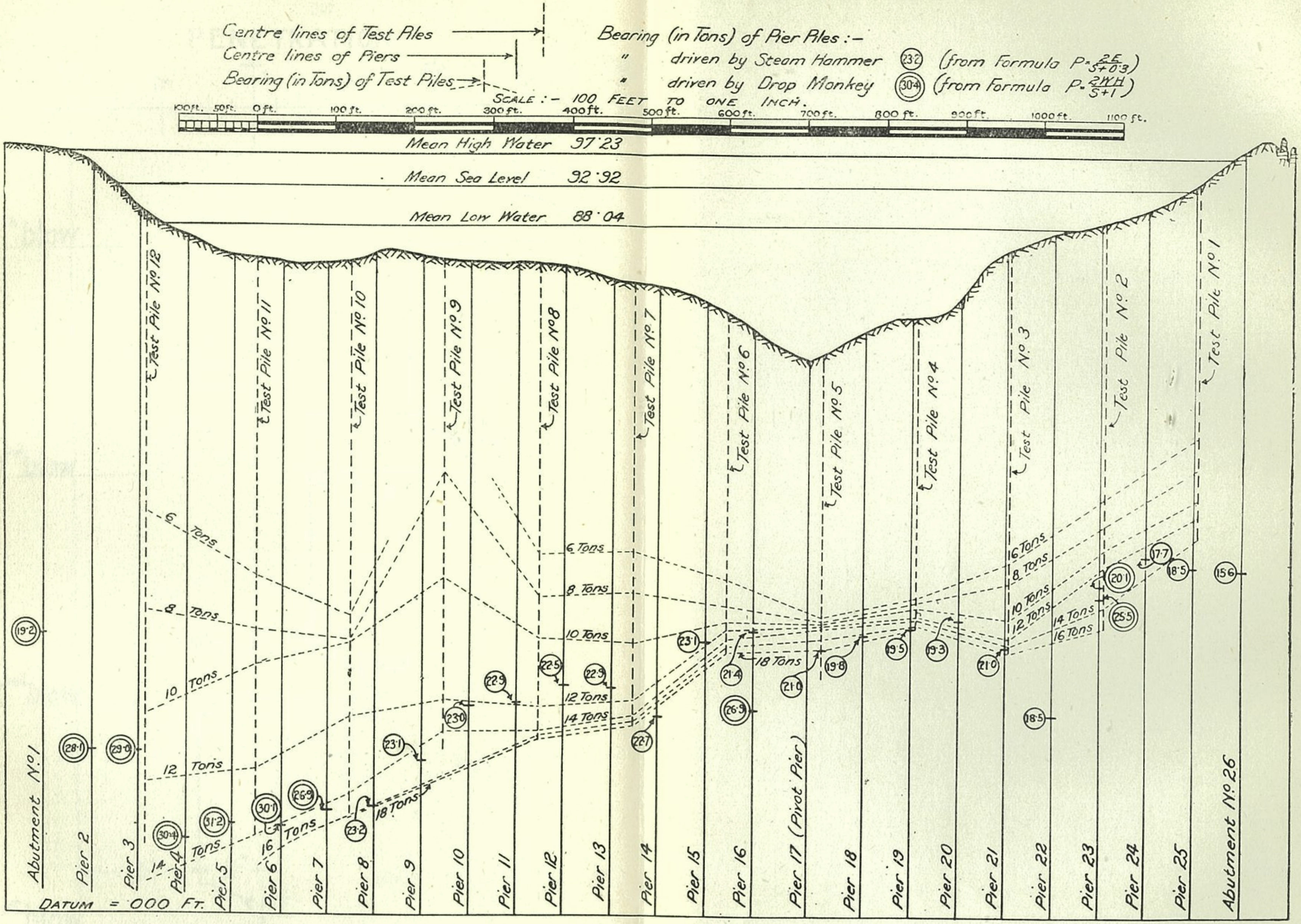
WAIHOU RIVER BRIDGE.

KOPU.

Graph showing relationship between Loading and Settlement.

FIG. 8.

Vertical scale: approx. 33′ to 1″ *Horizontal scale*: approx. 330′ to 1″



CROSS SECTION OF RIVER ON BRIDGE SITE, SHOWING BEARING POWER OF PILES DRIVEN.

WAIHOU RIVER BRIDGE. KOPU. FIG. 9.

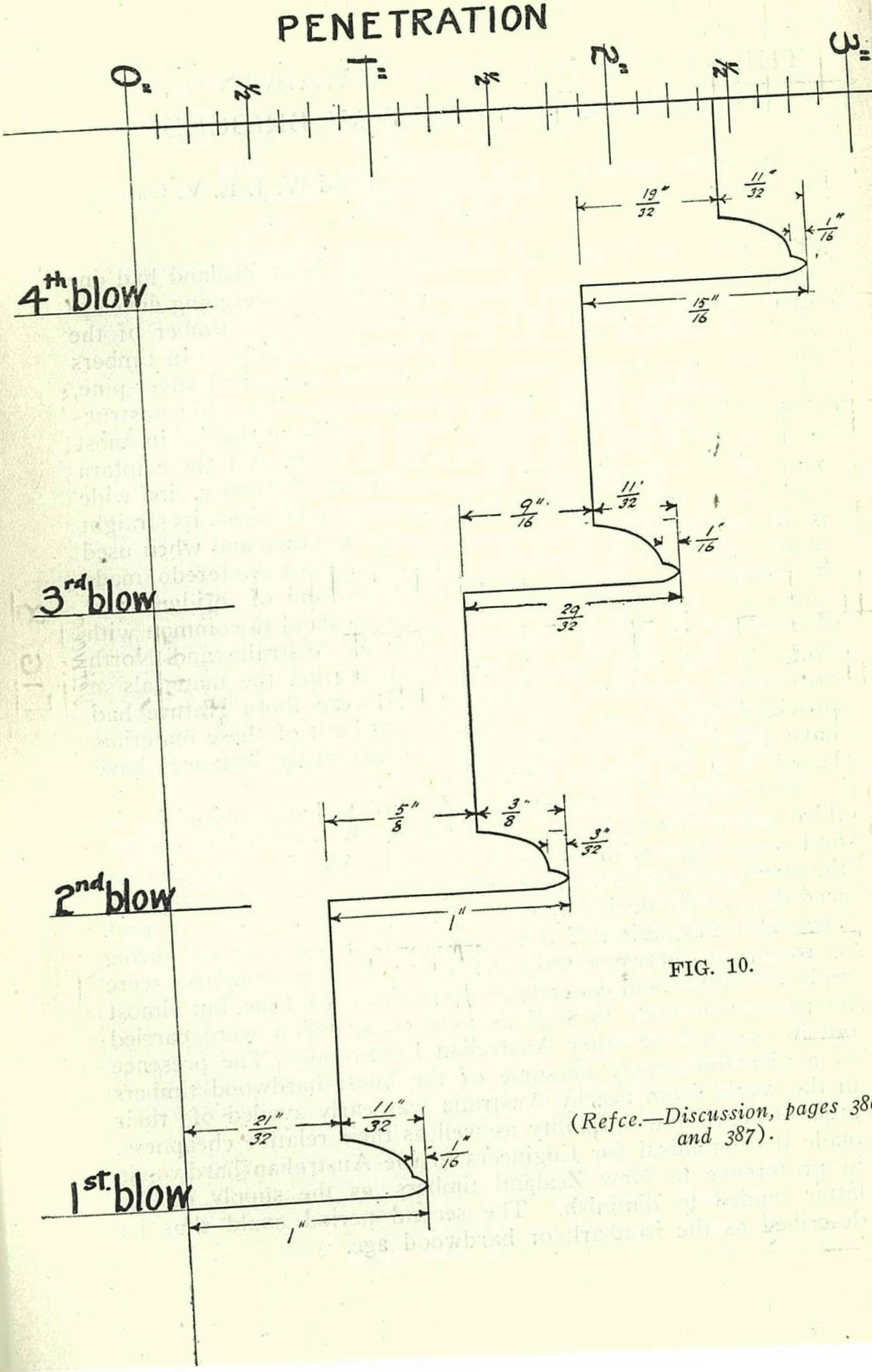


FIG. 10.

(*Refce.—Discussion, pages 386 and 387*).