Design Of The Main Foundations Of The Port Mann Bridge

Dr. H. Q. Golder, M.E.I.C.
H. Q. Golder & Associates Limited
Consulting Engineers
Toronto

G. C. Willeumier, M.E.I.C.
Vice President, C.B.A. Engineering Limited
Consulting Engineers, Vancouver

The glory of a bridge is the span. There is little glory in the foundations.* In the words of Coulomb in his famous essay on earth pressure, "Tandis que les grands hommes, portés au sommet de l'édifice, tracent et élèvent les étages supérieurs, les artistes ordinaires répandus dans les étages inférieurs, ou cachés dans l'obscurité des fondemens, doivent seulement chercher à perfectionner ce que des mains plus habiles ont créé."

This paper is written by "les artistes ordinaires---------cachés dans l'obscurité des fondemens." It describes those foundations without which there would have been no glorious span.

The Port Mann Bridge crosses the Fraser River upstream from the cities of New Westminster, and Vancouver, B.C. and is part of the Trans Canada Highway system. The choice of the site, the type of bridge, and the reasons for these choices were explained in G. W. C. Lake's paper "The Port Mann Bridge" which was published in the May, 1964 issue of The Engineering Journal.

This paper deals only with the design of the foundations at the site chosen, and principally treats the foundations of piers N1, N2, and S1 to S5.

**The authors find that they have inadvertently paraphrased part of a saying of Terzaghi's (1951) "On account of the fact that there is no glory attached to the foundations, and that the sources of success or failure are hidden deep in the ground, building foundations have always been treated as stepchildren and their acts of revenge for lack of attention can be very embarrassing."*

Geology & Soil Conditions

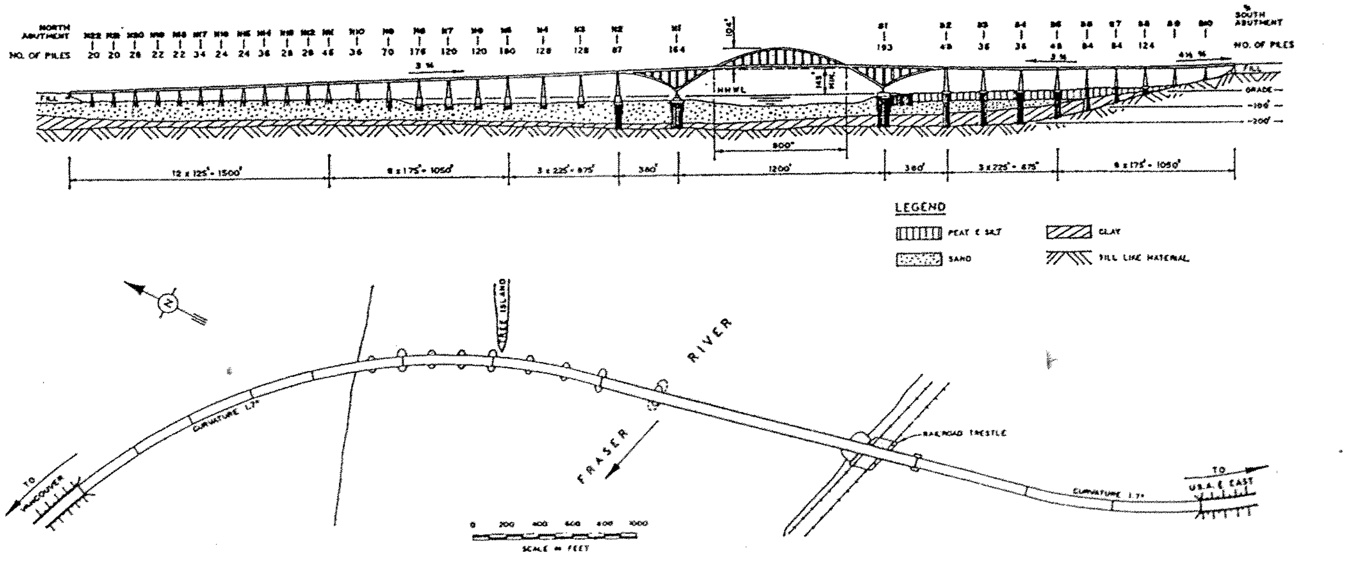
The site is in the lower flood plain of the Fraser River just upstream of the delta. The presence of a clay, which appears to be of marine origin, indicates that at one time the area was under salt water. The recent geological history of the region is complex and there have been advances and retreats of the ice sheets, changes in sea level, and probably isostatic rise in the land since the last retreat of the ice.[1, 12]

An investigation of the site was made using borings and static cone penetration tests. Samples were taken at regular intervals and in situ vane tests were carried out in the clay soils.

The soil profile along the centre line of the bridge is shown in Fig. 1a and typical boring logs are shown in Fig. 1b. On the south side of the river the soil conditions were worse than on the north side and consisted of a layer of soft peat to a depth of 15 ft. overlying soft organic silts and clay silts down to about 40 ft., below this again was a compact peat underlain by clay to a depth of from 45 to 50 ft. Layers of sand of varying density, with occasional layers of silt extended down to a depth of about 110 ft. and below this was gravel and sand to 120 ft. depth. From a depth of 120 ft. to 190 ft. the soil consisted of soft to firm sensitive clays and silts with occasional sand partings. Below this was compact granular material, some of which was till or till-like and some of which was probably waterlaid sands and silts which had been loaded by ice in the past.

Whatever the actual geological history of this material, for the purposes

Fig. 1a—Profile and Plan of Structure



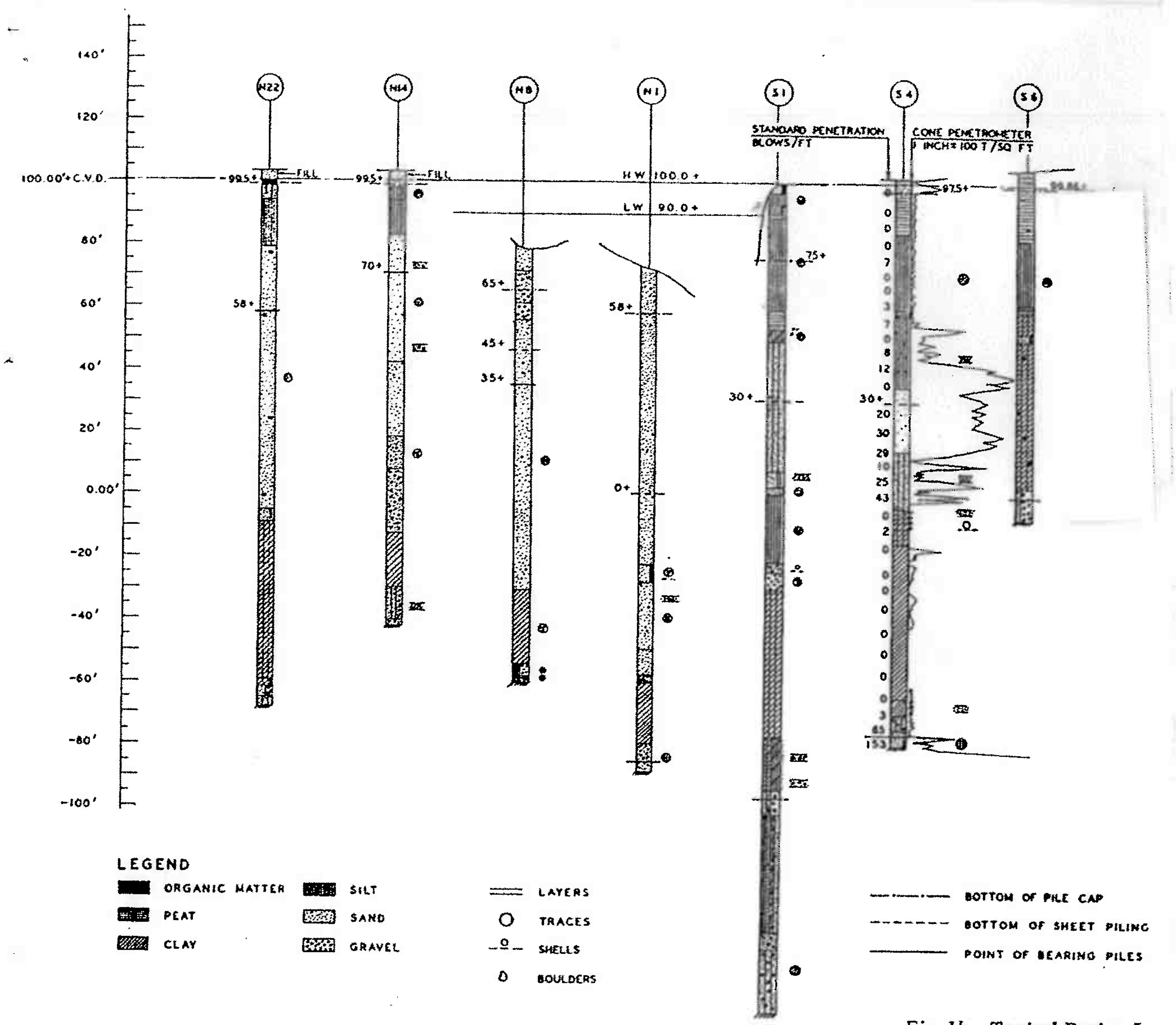


Fig. 1b—Typical Boring Logs

of this paper it is referred to as "the till" or "the till-like material". Artesian water pressure existed in some of the lower gravel layers. The foundation problem for the bridge stopped when the till-like material was reached.

On the north side of the river the conditions were simpler consisting of a thick sand layer overlain by some compressible material and overlying a clay layer of some 60 ft. thick. Below this was the till-like material.

The foundation problem was posed by the layer of clay from 120 ft. to 190 ft. deep on the south side. The shear strength of this clay, measured by in situ vane tests was about 2,000 p.s.f. Due to sampling difficulties shear strengths of about 1,000 p.s.f. were measured on samples in the laboratory. The sensitivity of the clay varied from 8 to 100, an average value being around 40. Consolidation tests showed compressibilities (C_c) in the range of 0.4 to 1.0.

Thus, the clay was sensitive to disturbances, not very strong, and compressible.[1, 8, 12, 13]

Foundation Problems and Principles of Design

The foundations had to be designed for the A.A.S.H.O. Standard loading conditions.

From the first it was obvious that the bridge could not be founded on the surface layers of peat and soft organic clay silt. What was not so obvious was whether or not the bridge could be founded in the sand layer at a depth of some 50 ft. A computation showed that the weight of the bridge and quite a modest fill placed on the south bank would cause a settlement of about 2 ft. due to consolidation of the clay layer at 120-190 ft. depth. The settlement on the north side would be much less. From the point of view of appearance alone such settlement could not be accepted. Either then, the vertical loads had to be carried to the till-like material, or the total loads must be reduced by eliminating all fill, and floating, or partly floating the foundation.

If the clay was disturbed in any way, say by driving piles through it, the settlement would be greatly increased by re-consolidation of the sensitive clay under its own weight plus that of overlying material.

Disturbance of the clay was therefore to be avoided. A further effect of settlement would be to add a downward frictional drag on the outside of any structure penetrating the clay down to the till.

Although, from the point of view of stresses in the bridge structure, a differential settlement of several inches could be accepted in the main span, and up to 2 in. in the subsidiary spans, the structure was sensitive to lateral tipping of the piers. This ruled out the use of floating foundations, and it was accepted as a principle of design that the vertical loads must be carried down to the till-like material. It seemed impracticable, however, to consider taking the horizontal loads down to 200 ft. deep, and some way had to be found to take these loads in the sand layer. Some provision would have to be made to accommodate possible settlement of the sand layer due to consolidation of the underlying clay. This fact ruled out the use of batter piles.

The above remarks apply to the main piers and to places where the soil conditions were worst. In other areas simpler foundations were possible.

The following principles of design for the main piers, i.e. N1 and N2, and S1 to S5 were therefore accepted:

(a) Vertical loads to be carried down to the layer of till-like material.

(b) Horizontal loads to be taken in the sand layer.

(c) General settlement of the whole area to be reduced to a minimum by avoiding the placing of fill.

(d) Provision to be made to accommodate settlement of the sand layer should it occur.

(e) Either, any drag on the structure penetrating to the till, caused by settlement of the clay to be included in the vertical load, or provision to be made to eliminate drag.

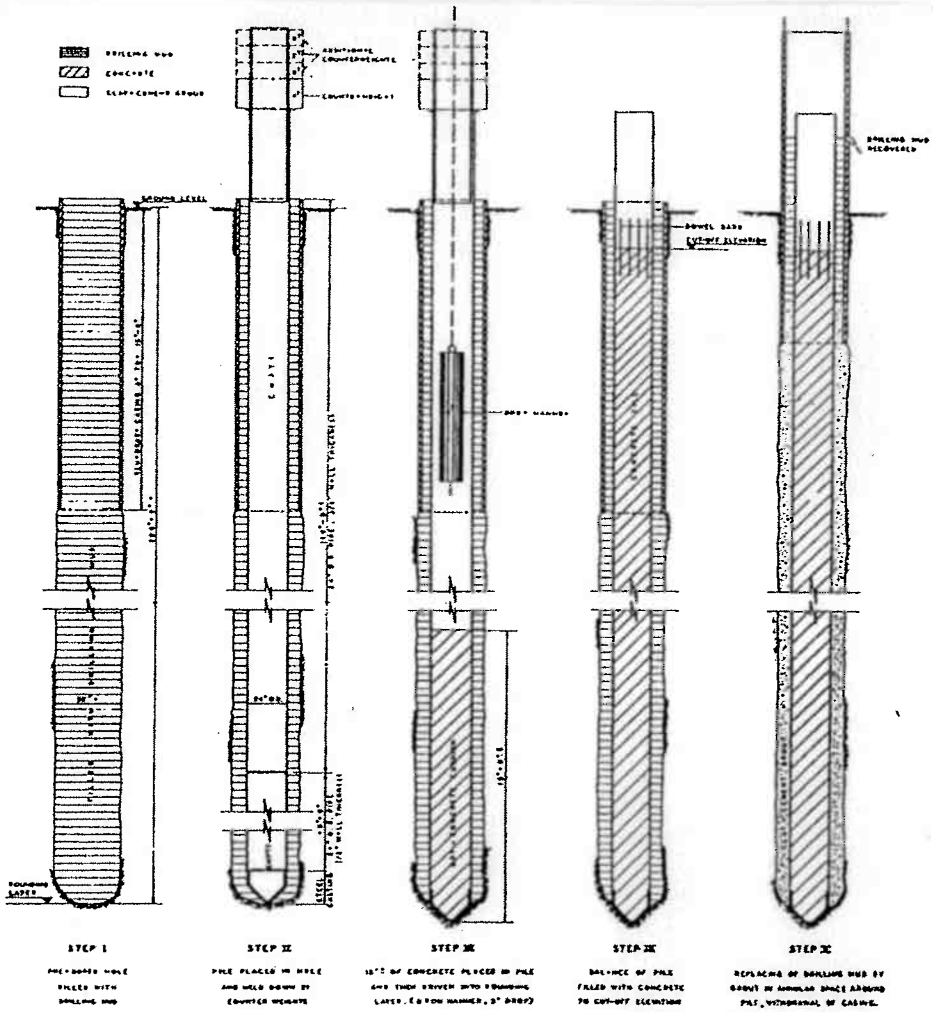


Fig. 2—Intended Method of placing a 200 ft. Bearing Pile

Possible Solutions

(a) *Caissons sunk to the till*

Many large piers for bridges have been built on caissons sunk through soft material to hard material. The practical limit of depth for compressed air work is around 100 ft., which rules out the use of compressed air at the Port Mann site.

Open caissons, sunk by grabbing, have been sunk to depths comparable to the 200 ft. required on this site, but as far as the authors are aware, never in quite the same conditions.

On two bridges in India[7, 16] and also in Australia[6] caissons were sunk to depths of 103, 160 and 158 ft. through soft silts, river mud and some sand. On the San Francisco-Oakland Bay Bridge[4] and the Tagus Bridge now under construction in Portugal[5] depths of 228 ft. and 260 ft. are claimed, but a substantial part of these depths is water, the depths through soil (soft sandy mud, some clay and sand) being 158 ft. and 170 ft.

At Port Mann the friction in the sand would be large and the clay is not soft, though sensitive. It might well have proved impossible to sink a caisson by normal means. Another difficulty is the uneven surface of the till-like material.

(b) *Large Diameter Concrete Cylinders sunk to the till*

Concrete cylinders 6 to 10 ft. diameter can be sunk to the till by grabbing. There will be considerable frictional forces on the outside of the cylinder, but these can be overcome by loading with ballast, a method impracticable in the case of a caisson.

There will be negative drag on the cylinders due to the settlement of the disturbed clay and this must be added to the applied load. Since the surface of the till is uneven, and as some ground will almost certainly be lost during the grabbing process, one cylinder may be undermined by an adjacent cylinder put down later.

(c) *Cast In Situ Bored Piles to the till*

Boring by a suitable technique would avoid practically all disturbances of the clay, but the possibility of drag must be included in the working load.

One of the difficulties of this method is assessing the allowable load on the piles in a variable material. A further disadvantage is that a bored pile will always settle more than a comparable driven pile even though the ultimate load may be the same.

(d) *Steel H-piles driven into the till*

If the foundation material had been rock, this would have been a good solution since the bearing value of an H-pile on rock is high. However, in a variable till-like material the bearing value of the thin section of a steel H-pile is not high. Also some drag must be added to the working load.

(e) *Precast Concrete Piles or Steel Pipe piles driven into the till*

The big objection to any displacement type of pile is that it would completely remould the sensitive clay causing large settlements and a downward drag on the pile in the order of 100 tons. In the case of concrete piles, there would be great handling difficulties.

(f) *Steel Pipe Piles placed in a Pre-bored Hole and then driven into the till*

The solution chosen was a composite

system which incorporates many of the advantages of the different systems considered above but avoids their disadvantages. As designed the system is as shown in Fig. 2 and is described as follows:

A hole is bored to the top of the till using mud, a steel pipe pile with a cast steel shoe and with the outside of the pipe coated with asphalt is placed in the hole and driven into the till and then filled with concrete. The mud surrounding the pile is displaced by a clay-cement grout.

This method avoids disturbance of the clay and since the piles are driven into the till, the driving resistance on each pile can be used in conjunction with load tests on a few piles to estimate the safe working load on the piles.

Description of Main Foundations—Piers N1-2, S1-5

The foundations of the main piers are designed as large pile-caps of injection concrete surrounded by steel sheetpiling from ground level to well into the sand layer (Fig. 1). This sheetpiling serves the dual purpose of a cofferdam for construction of the pilecap and of transfering horizontal forces to the sand layer. The foundations of piers S2-5 are designed according to the same principle except that the pilecaps are of normal concrete. The pilecap is supported by steel piles coated with asphalt and placed in pre-bored holes and then driven into the layer of till-like material.

(a) The Piles

The piles consist of steel pipes 24-in. outside diameter. The bottom section, 40 ft. long has a wall thickness of ½ in. and fitted with a cast-steel point. The upper sections have a wall thickness of ⅜ in. The greater thickness in the bottom section is to resist the tension stresses caused by driving with a drop hammer inside the pipe and to resist the external hydraulic pressure on the empty pipe when it is lowered into the mud filled hole.

The outside of the pipe is coated with an asphalt layer 0.2 in. thick. The properties of the asphalt are based on laboratory tests which indicated that with a thickness of 0.2 in. of asphalt, settlements slower than 2 in. per year would result in a maximum downward drag on the piles of 20 tons per pile. This was allowed for in the design. To prevent the asphalt coating from spalling during handling and driving of the pile, the layer is reinforced with fibreglass with the main strands encircling the pile. A secondary benefit is that the asphalt protects the piles against corrosion.

Boring the Holes

The holes are 8 in. larger in diameter than the piles to leave a 4 in. annular space around the pile.

It is important that in boring the holes the clay should be disturbed as little as possible, that the sides of the hole should not be allowed to collapse nor the bottom to blow even when artesian water pressures of as much as 10 p.s.i. at ground level were encountered. If casing is used in the top portion of the hole, and for purely practical reasons a length of say at least 15 ft. must be lined temporarily, then it must be removed as the grout was placed or if left in, must terminate well above the till and well below the pilecap. The method envisaged for the boring was the reversed circulation rotary method using mud. The method actually used is described in detail by Davie.[3]

In all, 612 holes had to be drilled, the total footage being about 100,000.

Driving the Piles

After completion of the hole a pipe pile was lowered into it. It was necessary to weight the pile during this operation to sink it. The inside of the pile was then filled to a depth of 15 ft. from the bottom with dry mix concrete. This concrete acted as a cushion for the drop hammer used inside the pile to drive it to a set in the bearing strata.

Since the piles were placed in a hole of slightly larger diameter and were therefore laterally unsupported when driven, and in view of their great length were very slender, the piles were likely to buckle if hit at the top by a driving hammer. The piles were therefore driven by dropping an 8 ton hammer inside the pile, hitting it near the point. To prevent the pile point from being separated from the pile, the energy of the blow had to be transferred to the wall of the pile. This was achieved by filling the bottom 15 feet of the pile with dry mix concrete.

It was specified that the piles were to be driven to a set of half an inch for 10 consecutive blows of 30,000 ft-lb. net energy.

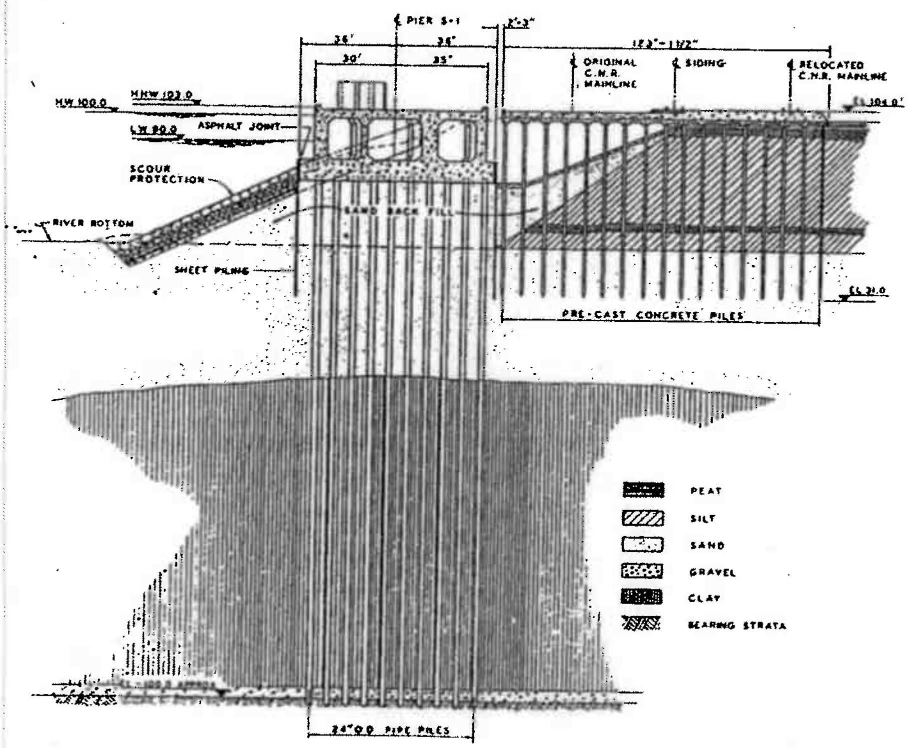
From previous cone-penetrometer tests, borings and laboratory strength tests, the design load of the piles was tentatively established at 150 tons per pile. The specifications were based on this value, but provision was made for a decrease in the number of piles, should the bearing capacity of the substrata prove to be higher than anticipated. Should, however, the bearing capacity of the substrata be lower than that assumed, the specifications provided for a means of improving this capacity by extending the prebored hole into the bearing strata and backfilling the extension with gravel before placing the pile.

This gravel, which was placed by gravity, was expected to form a bulb of strong material underneath the pile point, which would help in spreading the load in the subsoil.

The Clay Cement Grout

Due to their slenderness, the piles had

Fig. 3—Cross Section over Pier S1



to be supported laterally to be able to carry a reasonable load. The water or drilling mud filling the annular space around the pile had therefore to be replaced by a stronger material.

A very strong material was not desired, since this would act as an addition to the pile and transfer any frictional drag to the pile point. Ideally the material should have a shear strength somewhat lower than the shear strength of the surrounding soil, i.e., less than 2,000 p.s.f. The clay-sand-cement grout originally specified was found to be too strong and better results were obtained by leaving the sand out of the mix. A mix of 900 lb. of clay plus 40 lb. of cement gave shear strength between 1,500 and 2,500 p.s.f.

The grout was pumped into the annular space through a small diameter pipe lowered to the bottom and retracted with rising grout level. In addition to supporting the pile against buckling, the grout also prevented sand from penetrating the asphalt layer around the piles and destroying its flow characteristics.

Filling the Piles

Filling the top section of the piles with concrete was necessary to provide an adequate connection of the piles to the pilecap and to make the piles take a greater part in resisting horizontal forces in the piers.

Horizontal Forces

The horizontal forces on the foundations are resisted in four different ways.

(i) bending of the bearing piles.

(ii) passive resistance of the sand layer into which the steel sheetpiling surrounding the pilecap is driven.

(iii) bending of the steel sheetpile walls which are transverse to the direction of the horizontal force.

(iv) shear in the sheetpile walls which are parallel to the direction of the horizontal force. This shear is limited to the resistance to sliding in the clutches of the steel sheetpiles.

A major part of the resistance to horizontal forces comes from item (ii) above. In the case of pier S1 where the horizontal forces were large, and pier S5 where the forces were large and moreover the sand layer was thin, the peat above the sand was removed and replaced by sand in order to increase the passive resistance of the soil.

The part which each of the above items plays in resisting the horizontal forces is proportional to their stiffness. It would be dangerous to rely on item (ii) if the movement required to develop the passive resistance were greater than that which could be tolerated by the members involved in the other three items.

Relatively the worst horizontal loading condition occurs at pier S5 and is due to earthquake forces (which were taken at 5% of the weight of the structure in any horizontal direction). Certain limiting assumptions were made regarding the soil and the horizontal movement of the pilecap was calculated by letting each member (soil, sheetpiling and bearing piles) contribute according to its relative stiffness in resisting the load. The horizontal movement was found to range from ¼ in. to an extreme of 1½ in. This order of displacement can be absorbed by any of the members that are expected to offer resistance, without their being overstressed.

Since the resistance of a cofferdam to horizontal forces increases considerably with the compactness of the sand inside it, it was specified that the sand within the cofferdams of the main piers be compacted by vibroflotation over the top 30 ft.

Provisions for Vertical Movement

Since the sheetpiling surrounding the pilecaps was to be driven into a sand layer, which is subject to settlement, provision had to be made to prevent the transfer of possible drag on the sheetpiling to the bearing piles. This was achieved by a sliding joint between the sheetpiling and the pilecap.

The Main South Pier S1

The pier is situated in the riverbank and a railway track runs just to the south of the pier. This means that horizontal forces, additional to those occurring elsewhere, act on this pier, from the bank itself and from the rail track.

The rail track was relocated temporarily some 80 ft. further south to avoid interference with the construction of the pier.

To eliminate the horizontal earth pressure on the pier from the slope of the riverbank, the bank was excavated locally to the level of the pilecap as shown in Fig. 3. A reinforced concrete trestle was built across this excavation to support the railway tracks. The trestle was founded on precast concrete piles 70 ft. long which were driven into the sand layer.

At the location of Pier S1, the river bottom did not consist of sand only, as it was overlain by a layer of soft material; silt and peat, to a depth of about 25 ft. below the bottom of the pilecap. This material had to be excavated and replaced by sand which was then compacted by vibroflotation. When this deep excavation was carried out, a slip of the riverbank occurred, which necessitated some extra measures, such as driving of the southern sheetpile wall of the pier, which thus acted temporarily as a retaining wall.

The results of the pile load tests at the South Main Pier showed the desirability of reducing the design load on the piles of this particular foundation. The placing of additional piles was impracticable, since this would reduce their spacing to such an extent that interference of adjacent piles and prebored holes was feared. The weight of the pier itself was therefore reduced by redesigning it as a lighter, but much more heavily reinforced structure. The redesigned pier is a hollow box, divided into sections by a number of diaphragms, running in two directions. This reduced the load on the piles by about 20%. Provision was made to install a pump, should it be necessary to keep the inside of the pier dry.

Pile Load Tests on 24 in. Pipe Piles

As little or no experience existed with large diameter piles of the type used and on the layer of till-like material existing at the Port Mann site it was considered essential to check the working load of 150 tons/pile assumed in the design by loading tests on some of the piles. This working load was based on static penetration tests, exploratory borings with dynamic penetration tests, and laboratory tests on soil samples.

At the suggestion of the Department of Highways some of the test piles were equipped with strain gauges.

Method of Test

In all, 20 tests were conducted, Tests 1 to 4 were made on piles constructed specially for test loading and situated between the foundations. The grout surrounding these piles was a clay-sand-cement grout and the strain gauge readings on test pile 4 showed that the whole of the test load of 400 tons was carried by friction on the outside of the pile. No load reached the point. No further use was made of these test results.

In test pile 5 the grout was changed to a clay-cement grout to reduce the friction value, and similar grout was used in tests 7 and 11. In all the other tests a pure clay grout was used. Tests 5 to 20 were carried out on working piles in the pier foundations.

In principle the piles were test loaded to 500 tons, except those in the river piers which could only be loaded to 400 tons because of their unsupported length of about 30 ft. in water and the consequent danger of buckling. In some cases the settlement of a test pile indicated that the ultimate load had been reached at less than 400 tons.

The loads were applied by jacking against a steel beam and platform

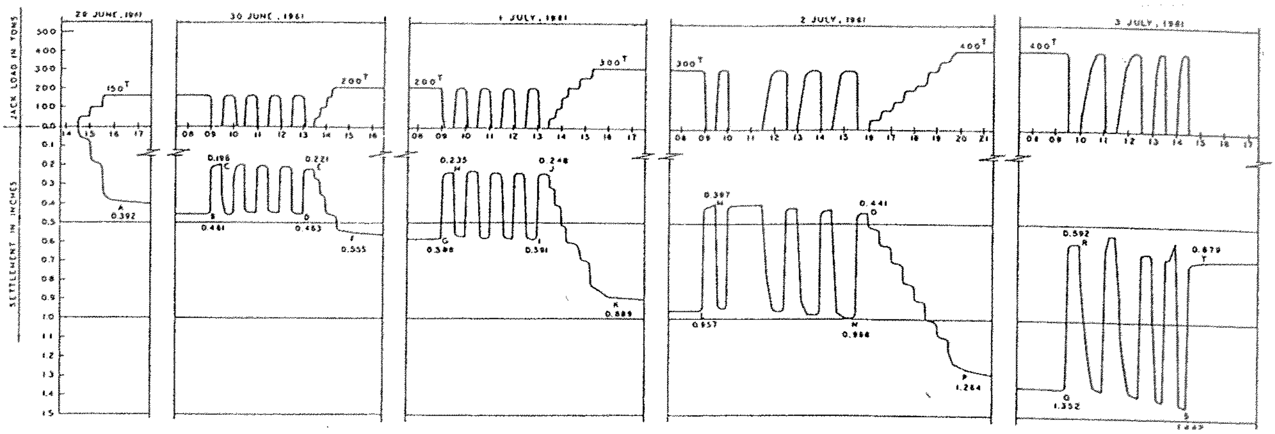


Fig. 4—Load-Time-Settlement Diagram of Test Pile No. 10 in Pier N1

loaded with ballast, the load being measured by a load cell and by the oil pressure in the jack. Settlements were measured by dial gauges and checked by direct readings using a precise level.

Each loading test took about a week. The first load applied was 150 tons. After the load had been in place for 19 hours, the load was removed and re-applied in half-hour intervals for a period of five hours. The load was then increased to 200 tons and after that increased in increments of 100 tons. All the increments were applied cyclically in the same manner as the first. Above a load of 300 tons the increments of load were made smaller if the settlements were large.

A typical load-time-settlement diagram is given in Fig. 4.

Strain Gauges

The strain gauge installation in the test piles was designed and supervised by Dr. R. F. Hooley of the University of British Columbia.[10]

Figure 5—Top Load-Top Settlement Diagrams of Typical Test Piles

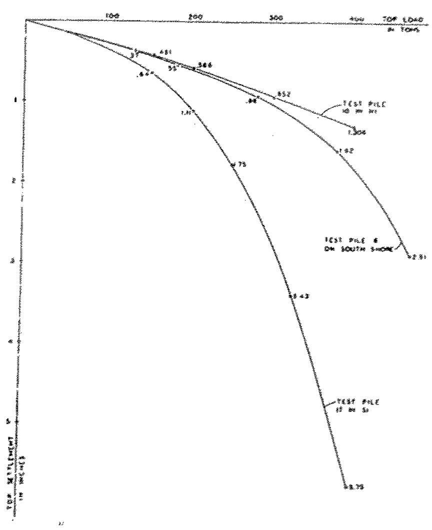


Table I—Loads and Settlement of Test Pile No. 10 — Pier N1

Load on Pile	*150ᵗ*	*200ᵗ*	*300ᵗ*	*400ᵗ*
TOTAL SETTLEMENT.........	0.463″	0.591″	0.998″	1.442″
0.463—0.461....................	—0.002″	—0.002″	—0.002″	—0.002″
0.591—0.588....................		—0.003″	—0.003″	—0.003″
0.998—0.957....................			—0.041″	—0.041″
1.442—1.352....................				—0.090″
Corr. settlement................	0.461″	0.586″	0.952″	1.306″
Elast. deformation...............	0.254″	0.348″	0.558″	0.761″
Point settlement.................	0.207″	0.238″	0.394″	0.545″
Point load......................	120ᵗ	170ᵗ	275ᵗ	370ᵗ

The results of the measurements showed that in test 7, in which a clay-cement grout was used, 35% of the applied load reached the point, and in tests 6, 9 and 14, in all of which a pure clay grout was used, from 60 to 80% of the applied load reached the point.

Interpretation of Results

A graph of load applied to the top of the pile against settlement of the top of the pile can be drawn directly from the test results. (Fig. 5).

In addition the method of cyclic loading adopted makes it possible to estimate the elastic movement of the pile and the skin friction acting on the pile by the method of van Weele[17] and thus to draw a load-settlement curve for the point of the pile.

Briefly this method is as follows:

(i) at any given load, the increase in settlement during cyclic loading is substracted from the total settlement. This gives the "corrected settlement" which is the settlement which would have occurred had the pile been loaded statically with the application of new increments at 20 hour intervals.

(ii) the elastic deformation of the pile plus soil, is taken as the difference in settlement under load and no-load during a cyclic loading period.

(iii) the load on the pile top is then plotted against elastic deformation. Under the higher loads this gives a sloping straight line plot .A line parallel to this line is drawn through the origin. The distance between these two lines, measured in the direction of the load axis represents the skin friction on the pile. This step assumes that the skin friction increases with load up to a certain value after which it remains constant. (Fig. 6).

(iv) the skin friction is subtracted from the load at the top to give the load at the point.

(v) the elastic deformation is subtracted from the "corrected settlement" to give the settlement of the point.

(vi) the point load is plotted against the point settlement (Fig. 7 and Table I).

The friction determined by this method can be compared with the values given by the strain gauge measurements given above. In the case of the clay-

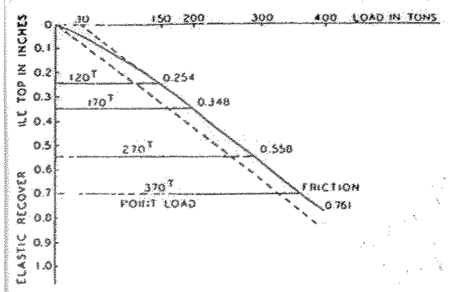


Figure 6—Method of Separating Point Load and Friction (Test Pile No. 10)

sand-cement grout no load reached the point. For the clay-cement grout the friction was about 66% of the applied load, i.e., 34% of the load reached the point, and for the pure clay grout an average of 75% of the load reached the point. It would be expected that in time most of the dead load will be transferred to the point as the surrounding clay consolidates under the load.

From the two plots of load-settlement, one for the top of the pile and one for the toe it is now necessary to decide on the safe working load for the pile. This is an empirical process based on experience. Several different approaches exist and these caused some disagreement amongst the engineers of the panel of consultants. Happily, and not unexpectedly, the final answers produced by each of the consultants did not differ greatly one from the other and there was no disagreement on the final design.

One approach is to estimate the ultimate load for the pile from the load-settlement curves and to divide this by a suitable factor of safety to arrive at a working load. If the load-settlement curve shows a clearly defined ultimate then the only decision to be taken is the factor of safety considered necessary. However, in some cases the ultimate load was not reached, and in others the load-settlement curve did not become parallel to the settlement axis of the graph although the pile had clearly failed. Judgement based on experience must be used in these cases. An apparently mathematical method, which may be useful in certain circumstances, but which is based on an empirical assumption[15] was used by some of the consultants, but was rejected as unsound by others. The ultimate load was divided by a factor of safety of 2 to give the working load.

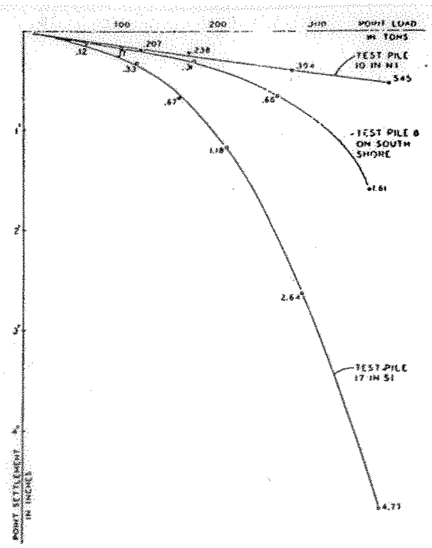


Figure 7—Point Load-Point Settlement Diagrams of Typical Test Piles

The other approach, which is probably fundamentally more sound, is to relate working load to allowable settlement. Here again the approach is empirical and based on experience. One criterion chosen was a settlement of the point of the pile of one inch. This cor-

Table II—Loading of Piles in Tons

Pier No.	Pile Test No.	Ultimate Load by Inspection	Working Load i.e. ult/2	Ultimate Load by v.d. Veen	Working Load i.e. ult/2	Working Load for 1" point settlement	Working Load for 0.2" cyclic settlement	Working Load for point settlement of 1" for increment of 150 T.	Design Load* (dead load +live load +wind or earthquake)
S5	5	570	285	500	250	160	240	210	
S3	6	570	285	350	175	300	220	260	
S2	7	>600	>300	500	250	200	—	230	
S3	8**	—	—	—	—	—	—	—	
S3	9	400	200	575	288	240	150	200	
	Average	535	267	480	240	225	203	225	150
N1	10	>600	>300	900	450	—	—	—	
N1	11	>600	>300	1000	500	—	—	—	
N2	12	>600	>300	900	450	—	—	—	
	Average	>600	>300	930	467	—	—	—	150
S1	13	280	140	205	103	170	125	110	
S1	14	250	125	325	163	120	135	60	
S1	15	250	125	400	200	130	135	55	
S1	16	350	175	300	150	265	190	220	
S1	17	280	140	325	163	195	170	140	
S1	18	280	140	325	163	180	145	130	
S1	19	550	275	400	200	—	—	—	
S1	20	280	140	220	110	135	145	80	
	Average	315	156	310	155	171	149	113	122

NOTES: *The figures in this column do not include a possible additional load due to drag of the soil.
**Testpile No. 8 was cancelled.

responded approximately to a settlement of the top of the pile of from 1.4 to 1.9 in.

A further criterion taken arbitrarily was the load which caused an increase in settlement of 0.2 in. under cyclic loading.

Finally the working load was taken as that load which caused a rate of increase of settlement of one inch per 150 ton increase of load. This can be expressed as a certain slope of the load-settlement curve.

When the working load is defined directly in terms of settlement, the term factor of safety has no meaning since the determination of the ultimate load has been bypassed.

Table II gives working loads for typical pile tests arrived at by the above methods.

The settlements which occur in the pile load tests are only indirectly related to the settlement of the pier itself. Estimates of the settlement of pier S1 were made, but this was a difficult and complicated problem which will be published elsewhere. By January, 1964, pier S1 had settled 1⅞ in. under its own weight and an additional 1 in. since erection of the superstructure.

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