THE LIONS' GATE BRIDGE

By

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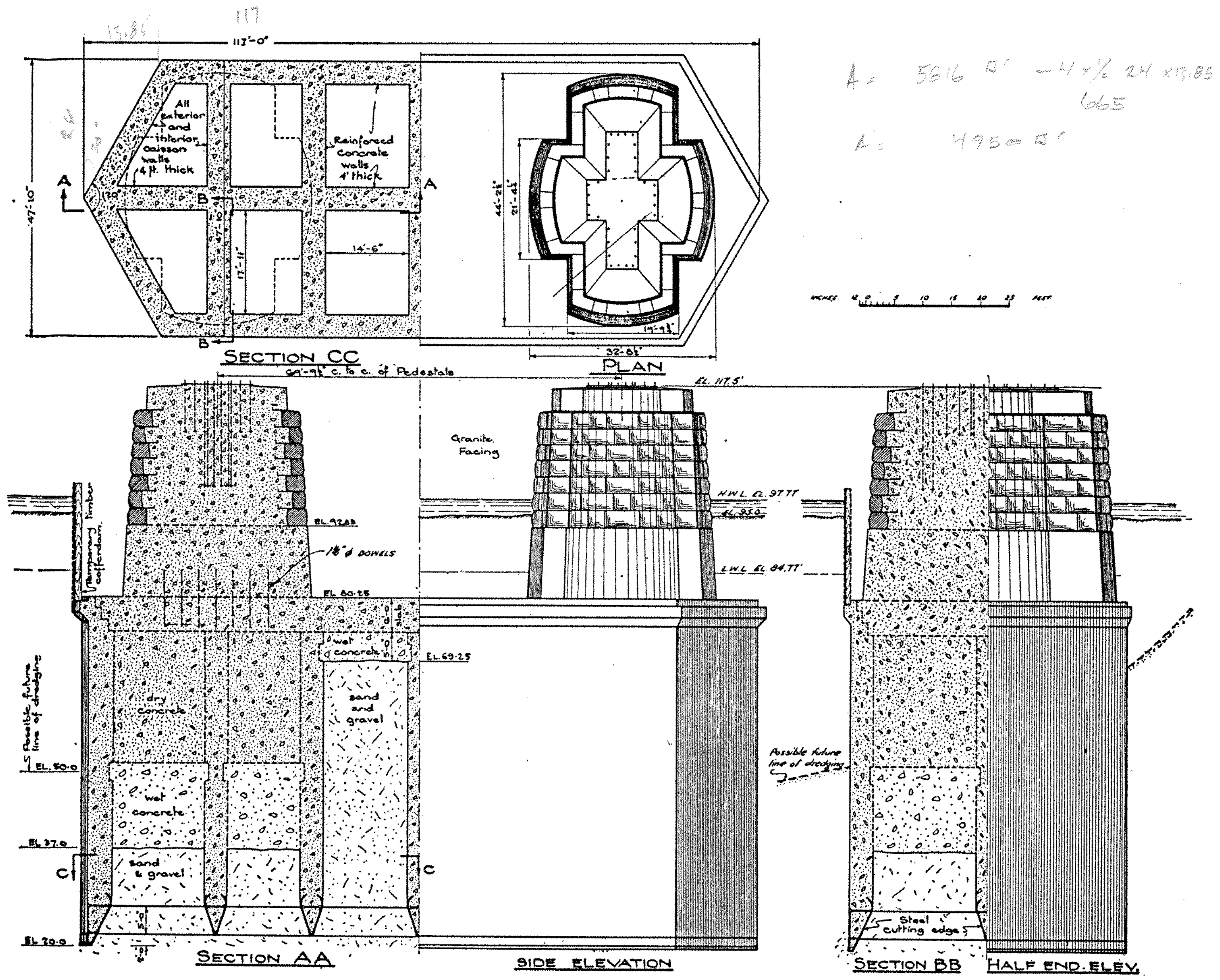

Fig. 12—North main pier.

1937, the excavation proper starting on July 19th of the same year, but the last details of the plaza were not completed until the end of January 1939, some three months after the opening of the bridge to traffic. Concrete for the main part of the pier was hauled from the mixing plant on an inclined railway which passed underground through the bluff at the lower part of the hillside. Later concrete, for the road slab and buildings, was brought ready-mixed from Vancouver. As for other parts of the substructure, a dry mix was always used, and vibrated into place. All concrete for the south anchorage was required to yield a strength of 2,500 pounds per sq. in. at 28 days.

North Main Pier

The pier which carries the north main tower is situated on shore, above the level of all but high tides. The ground of the north shore is formed of a hard-packed coarse wet gravel, laid down as a deltaic deposit by the Capilano River. This gravel extends to a considerable depth, overlaying the sandstone on which the south main pier rests by about 300 ft., as is evidenced by borings made at the time of construction of Greater Vancouver Water District's pressure-tunnel under First Narrows. The depth and size of this pier were influenced by the extremely remote possibility that the gravel on its south side may be dredged away to Elevation 50 in an endeavour to extend the navigable channel. As the site is at present, scour is non-existent, and the pier is considerably deeper and heavier than is necessary for stability.

The pier footing consists of an open cellular caisson with outside dimensions of 48 ft. and 117 ft. (Fig. 12), the shape being adopted in view of the possible future dredging. The steel cutting-edges (forming the toes of the walls and the partitions, all of which are 4 ft. thick) were assembled on the site after this had been levelled-off at Elevation 92.5; and sinking of the caisson was begun as soon as the reinforced concrete incorporating the cutting-edges had been poured to a height of 7½ ft. The sinking process consisted of alternately dredging inside the pockets (by orange-peel buckets operated by two derricks mounted on piles) and building-up the walls (see Fig. 13) until the cutting-edge had penetrated to the designed level at Elevation 20. A temporary timber cofferdam was built on to the top of the peripheral wall for use during the last stages of sinking and for construction of the pier-shafts in the dry.

The caisson followed the excavation under its own weight for the first 50 ft., after which sinking was accomplished almost entirely by excavating below the cutting-edges and then destroying the skin-friction by the explosion of light charges inside the dredging-pockets, a total of about 200 lb. of 40-per-cent dynamite being used for this purpose. As a matter of incidental record, the skin friction when the caisson was within a few inches of its final set and entirely unsupported by the cutting-edges was computed to be about 450 lb. per sq. ft.; though there is no evidence that this friction would have maintained the pier in its otherwise unsupported condition in-

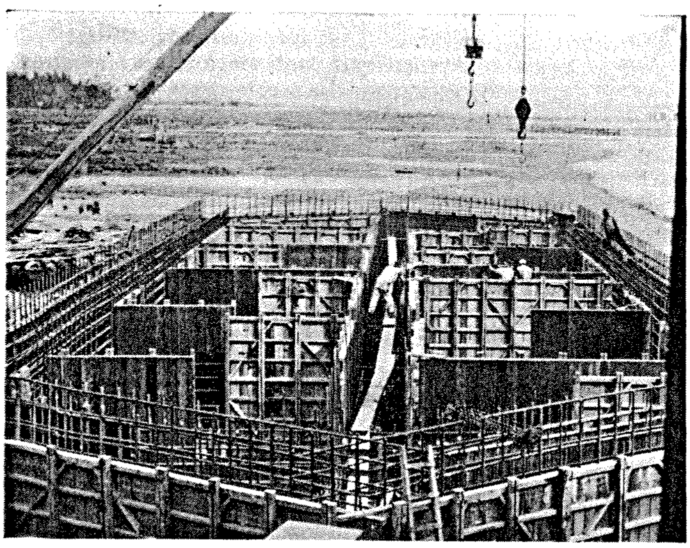

Fig. 13—North main pier; ready for wall-concrete

definitely. The sinking of the caisson occupied a period of three months (August to November 1937), at the end of which the cells were backfilled and concreted as shown in Fig. 12. The operation took place without incident beyond the appearance of several vertical hair-cracks at the top of the 7½-ft. wall, at the first sinking. This condition was rectified by the placing of extra longitudinal wall reinforcement at the bottom of the adjoining pour. The level of the caisson was readily maintained by regulation of the sequence of excavation of the various pockets in accordance with circumstances.

Upon unwatering the timber cofferdam, it was found that the upper 6-ft. rim of the caisson had cracked in a number of places. This top part of the wall, in order to accommodate a solid 6-ft. capping slab, was thinner than the main wall below and was unsupported by the cross-walls; and, since the cracking did not extend into the main wall, it was not considered to be of significance. The cracks were repaired by sealing on the inside with quick-setting cement, and the pier footing was completed by the pouring of the 6-ft. slab over all the pockets, its top surface being at Elevation 80.25.

The pier shafts differ from the south ones only in that they extend 3 ft. deeper and in that the ashlar facing, serving no useful purpose below shore level, is discontinued immediately below that elevation. The elevation of the precisely-dressed caps is 117.5.

Two strengths of concrete were specified. That for the cutting-edges, for the top slab of the footing, and for all work above the footing was required to develop a strength of 3,000 lb. per sq. in. at 28 days; while 2,500-lb. concrete was used for the caisson walls and for backfilling in the pockets. In order to expedite the work towards the end, the pier shafts were capped, as on the south main pier, with concrete containing high-early-strength cement. Slumps varied from 1½ in. for the shaft-tops to 3½ in. for the wall concrete and as much as 6 in. where the concrete had to be worked into the steel of the cutting-edges. Concrete was supplied from a mixing-plant (equipped with two one-yd. mixers with weighing-batchers and a controlled water supply) built nearby, and all material was placed by two timber derricks erected immediately north and south of the site.

Work went forward simultaneously with that on the south side, being started on April 9th, 1937 with the construction of plant. Erection of the cutting-edge began on July 5th, and the pier was finished on February 25th, 1938.

Bearing-pressure under the pier was at first concentrated largely on the cutting-edges. As settlement occurs, however, owing to the compressibility of the backfill under the concrete plugs of the excavation chambers, the variable pressures at the level of the cutting-edge will tend to reduce to a general average load over the whole base plane; while simultaneously the bearing pressures underneath the concrete plugs of the dredging chambers will increase from a small initial quantity to their final value (which depends on the elevations of the plugs above the bottom of the footing).

From experience of this same gravel-bed at Second Narrows Bridge, six miles further up the Inlet, the engineers anticipated that, during the above-described adjustments of bearing pressures, the pier would settle approximately 1¼ in. over a period of two or three years, and that the greater part of this settlement would occur during the superstructure erection. The pier top was therefore dressed to an elevation higher than the theoretical by that amount. In Fig. 14 the progress of the settlement over the first year has been plotted. Subsequent indications are that the settlement, though rather more than was expected, is tapering off satisfactorily.

Figures for the final average pressure are as follows, the conditions for maximum effect (i.e. low water and greatest superstructure load) being taken. Loads are stated in kips.

Pier:		
Cutting-edge 90, Reinforcing steel 270...	360 kips	
Concrete	41,200	
Gravel backfill (wet) in pockets (122 lb./cu. ft.)	10,760	
Gravel backfill around pedestals	4,840	
Granite facing (170 lb./cu. ft.)	2,340	
		59,500 kips
Superstructure (D.C.T. as for south main pier)		11,670
Total		71,170 kips
Less: Resistance from skin friction (on footing only) at 400 lb./sq. ft.	6,990	
Buoyancy at low water	19,980	26,970
Net total		44,200 kips

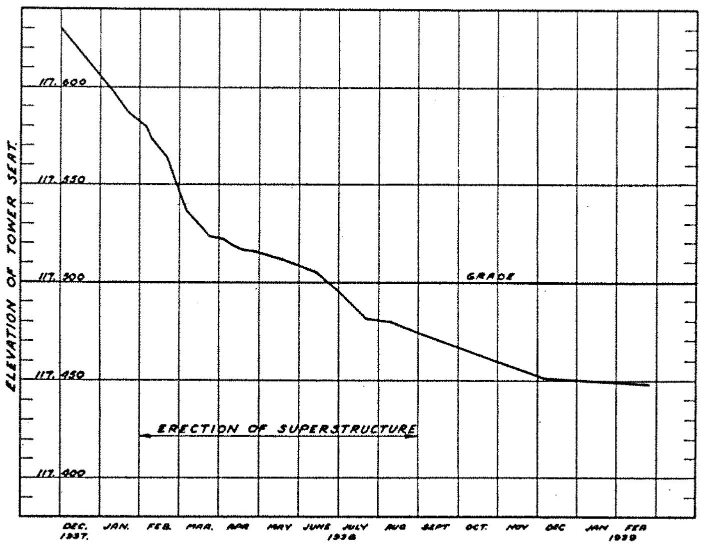

Fig. 14—North main pier: progress of settlement.

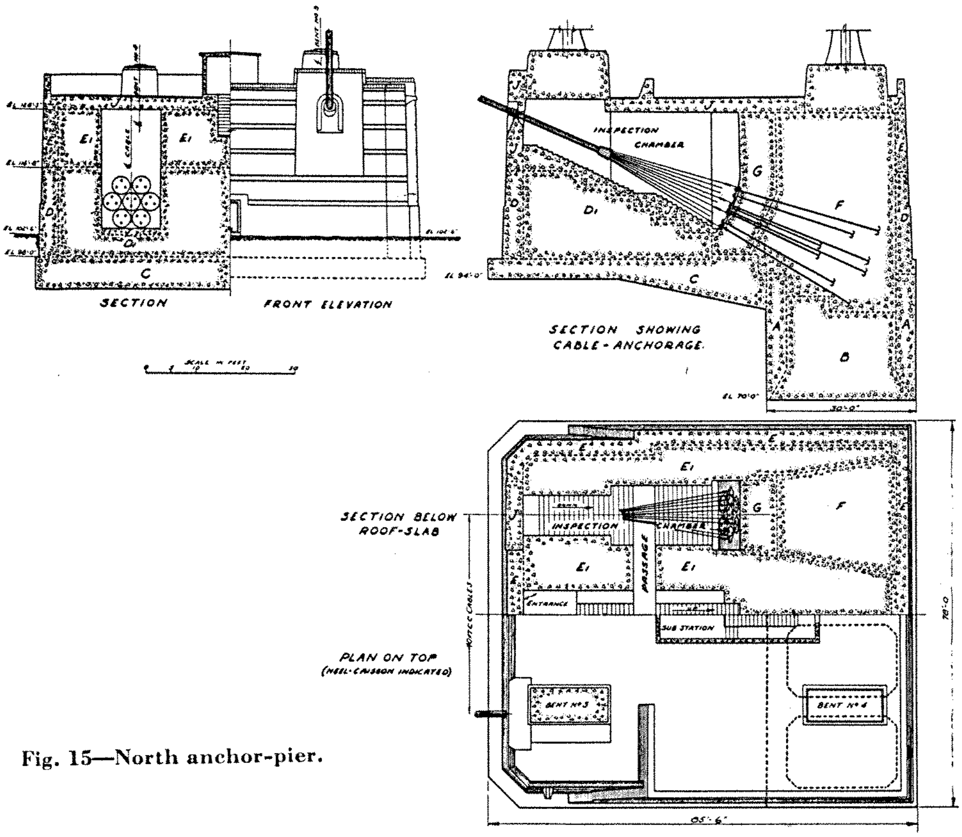

Fig. 15—North anchor-pier.

The base area of the caisson being 4,936 sq. ft., this gives an average bearing pressure over the horizontal plane at Elevation 20 of 9.0 kips per sq. ft.

At the same time, the natural pressure of the gravel on the same plane is made up as follows, figures being in pounds per square foot.

From 11 ft. of "dry" gravel above low water level, 11 x 100	= 1,100
From 64 ft. of wet gravel (with approximately 40% voids) below low water level, with allowance for buoyancy of same, 64 (122 - 64)	= 3,712
	4,812

The ultimate "punching" pressure over the area of the base plane is thus 4.2 kips per sq. ft.

North Anchorage

Conditions on the north shore, together with the fact that a large structure above ground would not be objectionable (there being no prospect of any development of the low-lying marshy terrain of the delta) led to the choice of a simple gravity-type of anchor-block at this end. The pier is positioned so that it is utilised to support one of the viaduct towers (thus reducing the tonnage of steel in the viaduct and also employing some of its weight as kentledge), while at the same time the slope of the backstay of the main cable is very close to the optimum for economy. The pier is shown in Fig. 15.

Basically, the pier consists of a heavy concrete box containing an anchorage for the main cables, together with suitable inspection chambers and means of access. The pedestals for two of the viaduct bents, and also a sub-station whence the electric supply for the bridge is distributed, are carried on the flat roof, which is surrounded by an ornamental parapet wall 4 ft. high. The exterior of the pier is treated architecturally (see Fig. 16), the dominating features being the provision of a pronounced set-back in each side wall normal to the direction of the cable, and the interruption of the large vertical concrete faces with horizontal rustications. The tops of the parapet walls have a fluted finish, pre-cast gargoyles are provided to throw the roof drainage clear of the walls, and arched niches are employed at the points of egress of the cables. Access to the pier is provided by a door at the bottom of the front face, and a stair connects with the cable-inspection chambers and with the pent-house sub-station, where a door leads out onto the roof. Communication with the viaduct deck may be made by a ladder running up the side of one of the bents. The inspection chambers are ventilated by concrete grilles set low down in the side walls.

At the rear of the pier, and extending over the full 78-ft. width of the structure, is a heel block 30 ft. wide and sunk 24 ft. deeper than the general base of the "box." This was provided in order to concentrate dead weight where it is needed, and also to present a positive resistance to sliding. Construction began with the laying-down of the cutting edge of a four-pocket open caisson for the heel block. The steel-shod shoes were filled with 3,000-lb. concrete, while 2,500-lb. concrete was used for the caisson walls (denoted by the letter "A" in the figure) and for all other work in this pier. No difficulty was encountered in sinking the caisson, after which the cells (the walls having been roughened to provide maximum bond) were sealed with tremie-concrete to a depth of 12 ft. and built up to the shape denoted by "B." Excavation for the base slab in front of the heel then went forward and this slab "C" was the next to be poured. The outside walls "D" were then built to a construction plane at Elevation 115.67, after which the large mass of concrete "D_1" was deposited, leaving cavities for the wedge-shaped masses incorporating the anchorage-steel. Then followed walls "E" to the elevation of the bottom of the roof-slab, and the remaining mass-concrete "E_1," around the anchor cavities. At this stage, the superstructure contractor assembled the cable anchorages, which were then concreted in: the anchor masses "F" each contain about 670 cu. yd. of concrete. Pedestals for the viaduct were then built, those for Bent 3 being carried on shallow concrete beams over the cable chambers. During the building of the pier, all construction joints were thoroughly cleaned of laitance

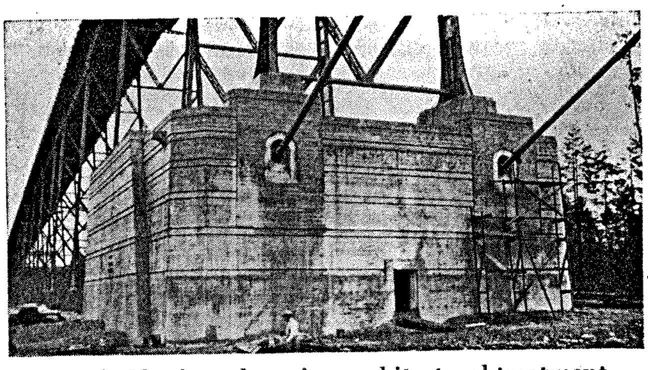

Fig. 16—North anchor-pier: architectural treatment.

and roughened, important bonding surfaces being heavily reinforced. Vertical wall joints were sealed with copper water stops.

The pier remained in the unfinished state described (i.e. with the roof slab and parapet walls and the upper part of the front wall missing: see Fig. 17) during the assembly of the main cables, the anchorage chambers being open to facilitate unreeling of the strands and assembly of the sockets.

After the cables had taken up their final normal position and the backstays had been wrapped, construction was completed with concrete work "J," which comprises the waterproofed roof slab, parapets, the concrete around the cable entrances, the sub-station, and the stairways. The ground around the pier was backfilled to Elevation 102.5.

Calculations concerning stability were made for all stages of construction, the pours of concrete being so arranged chronologically as to obviate the occurrence of unduly large bearing pressures and dangerous sliding conditions. Sliding resistance was computed separately for the two horizontal bottom surfaces, for the vertical front of the heel block, and for the inclined base of the central slab, the angle of friction of the gravel being taken as $\tan^{-1}.40$. The greatest tendency to sliding occurs at high tide, with lateral wind (but no live load) on the approach bents and with maximum cable pull, and for this condition the horizontal component of cable tension is 11,556 kips, while the resistance of the pier to sliding is computed as 24,902 kips. The maximum toe pressure under the pier occurs also with the greatest cable pull, but at low tide and with both live load and wind on the viaduct, and is 6.08 kips per sq. ft. The maximum under the rear edge of the heel caisson took place during construction, immediately before assembly of the cables, and amounted to 7.65 kips per sq. ft. Under working conditions the heel pressure does not exceed 5.40 kips per sq. ft.

The amount of concrete in the pier is about 9,780 cu. yd.; and its total weight, including cutting-edge (13 kips), reinforcing steel (219 kips), and cable anchorage assembly (134 kips), but not including the electrical substation and equipment, amounts to approximately 20,250 tons.

Construction of the pier began on July 6th, 1937, and it was ready to receive the anchorage steel in November of that year. The final pour of concrete was made on November 4th, 1938. All concrete was supplied from the mixing-plant near the main pier.

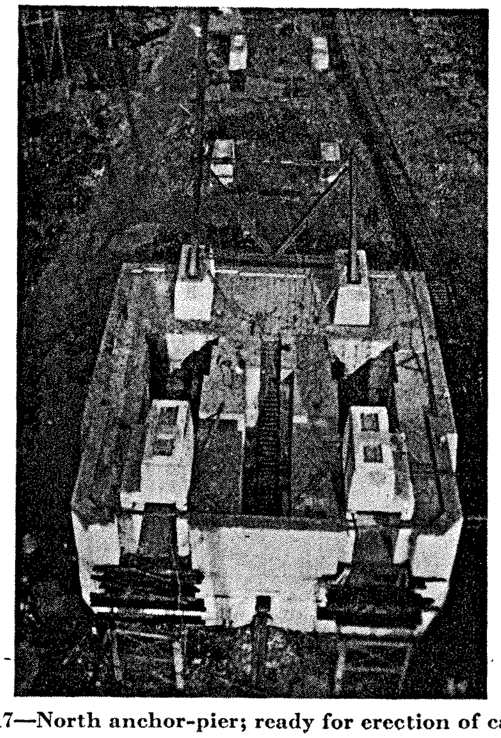

Fig. 17—North anchor-pier; ready for erection of cables.

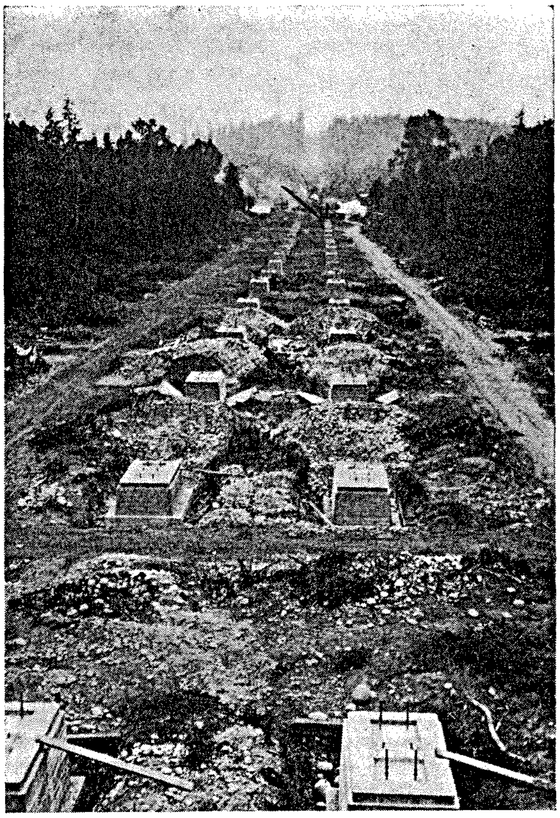

Fig. 18—Pedestals for north viaduct.

Viaduct Substructure

The girder-spans of the north viaduct are supported (see Fig. 2) on 24 steel bents, a concrete abutment at the north end, and the cable-bent at the end of the side-span. There are twenty-five pairs of concrete pedestals, those which carry Bents 3 and 4 forming part of the anchorage pier, and the remaining 46 being founded on the gravel formation that has already been described. The pedestals have appropriate spread footings which are taken to a depth that depended on the local conditions in each instance. For the footings, 2,000-lb. concrete was specified, and 2,500-lb. concrete for the shafts. The maximum bearing pressure does not exceed 2.6 tons per sq. ft. except in the cases of the most northerly bent (where, owing to the stiffness of the short column, the pressure may attain an extreme value of 3.7 tons per sq. ft.) and of the heavy cable-bent (where cable pull and side-winds render 5.6 tons per sq. ft. possible, under the deeper western footing). Anchor-bolts for the steel were set into the concrete, but the bearing surface of the pedestal was not dressed to elevation except for a small central area. On either side of this bush-hammered portion were left 6-in. recesses that were grouted up to the base plates after the bents had been plumbed. In the way of aesthetic treatment, the sides of the pedestals are battered at a slope of 1 in 16, and the upper 8 in. of the sides are set back 3 in., the 8-in. face being painted with black asphaltic paint. A general view of the viaduct pedestals is to be seen in Fig. 18.

The abutment, at the south end of the embankment, consists of a base slab 57 ft. by 20 ft. and 3 ft. thick, on which are carried four columns in the form of longitudinal counterforts (Fig. 19). The two principal coulmns at 22-ft. centres support the bridge seats, and the two outer columns at 50-ft. centres support the ends of the breast

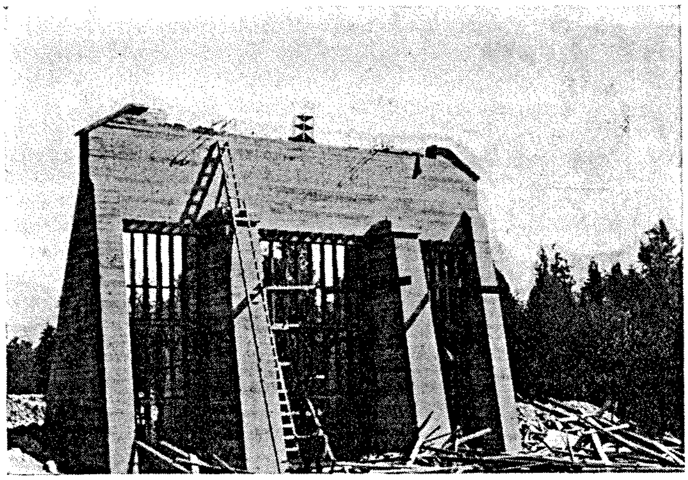

Fig. 19—Open abutment at north end of viaduct.

wall. This vertical wall is 18 in. thick and 12 ft. deep, suitably reinforced, and serves to retain the fill above a point about 2 ft. below the elevation of the bridge-seats, the remaining depth of the fill being allowed to spill around and through the abutment: the top of the wall is shaped to the profile of the roadway. The height of the abutment from grade to footing is about 40 ft., this figure deriving from study of the comparative costs of embankment and steelwork, and of the permissible weight and lateral extent of the former.

About 3,400 cu. yd. of concrete were required for the pedestals and abutment. This part of the job, including excavation, proceeded concurrently with that of the remainder of the substructure, being started in April 1937 and completed in time to receive the steelwork in November of the same year. No frost was experienced during construction or in the period of curing. Concrete was supplied from the mixer at the north main pier, and distributed by truck.

North Embankment

As may be seen from Fig. 2, the length of the main embankment, extending from the end of steel to the south abutment of the Marine Drive overpass, is approximately 1,000 ft.

For descriptive purposes, it may conveniently be divided into three parts. Over the first section, extending some 340 ft. northward, the roadway continues downward at the grade of the viaduct, and retains its normal 37-ft. overall width. The next section, extending (with a length of 400 ft.) as far as the division of the road into the three traffic ways comprising the junction with Marine Drive, carries the eight-lane toll-collection plaza. A 100-ft. vertical curve reduces the gradient to 0.25 per cent and, in a distance of 220 ft., the roadway widens to the full plaza width of 109 ft. between kerbs, this width being maintained for the remaining 180 ft. of the section (see Fig. 20). Drainage over the plaza is effected by a four-inch parabolic roadway crown,* assisted by the slight longitudinal gradient; catch-basins and drains are provided. The toll station itself (Fig.21) consists of three double-lane toll booths, and one single-lane booth on the west side of the road. On the east side stands the administration building which, together with its garage accommodation, covers an area of 31 ft. by 82 ft. The three main toll booths each consist of a small enclosed, heated, concrete building mounted on a concrete "island" 47 ft. long and 9 ft. in maixmum width, tapering nearly to a point at each end, where a concrete protection post is set up. The kerbs of these islands are four inches high for the greater part of their length, but increase in height towards the ends to give protection to the booths proper. The two traffic ways between the main booths are sheltered by a common canopy of concrete; but the outer ones, in order to admit high truck loads, are uncovered.

A covered service-trench (for electrical conduits, steam pipes, etc.) connects the series of booths with the administration building; and cavities, covered by removable slabs, have been left in the roadway to permit of the future installation of electro-mechanical traffic-registering gear should the use of such become advisable. The administration building is equipped with water supply, and a septic tank is provided for disposal of the sewage effluent.

The third section of the fill consists of foundations for three diverging roadways. The westerly side-road, forming the inlet from West Vancouver and the popular Whitecliff highway, is 24 ft. in width. That on the east side, being the comparatively little-used outlet towards North Vancouver, is 13 ft. wide as built, but may be broadened in case of necessity. These two outer roads lead directly into Marine Drive on gentle downward gradients. The central road, leading from the plaza on an up-grade of 3.6 per cent, is 24 ft. wide, accommodating traffic both from the east (North Vancouver) and to the west. Traversing Marine Drive by a 40-ft. rigid-frame concrete overbridge, this road veers to the east, and, descending on a gravel fill, divides into two single-lane tributaries.

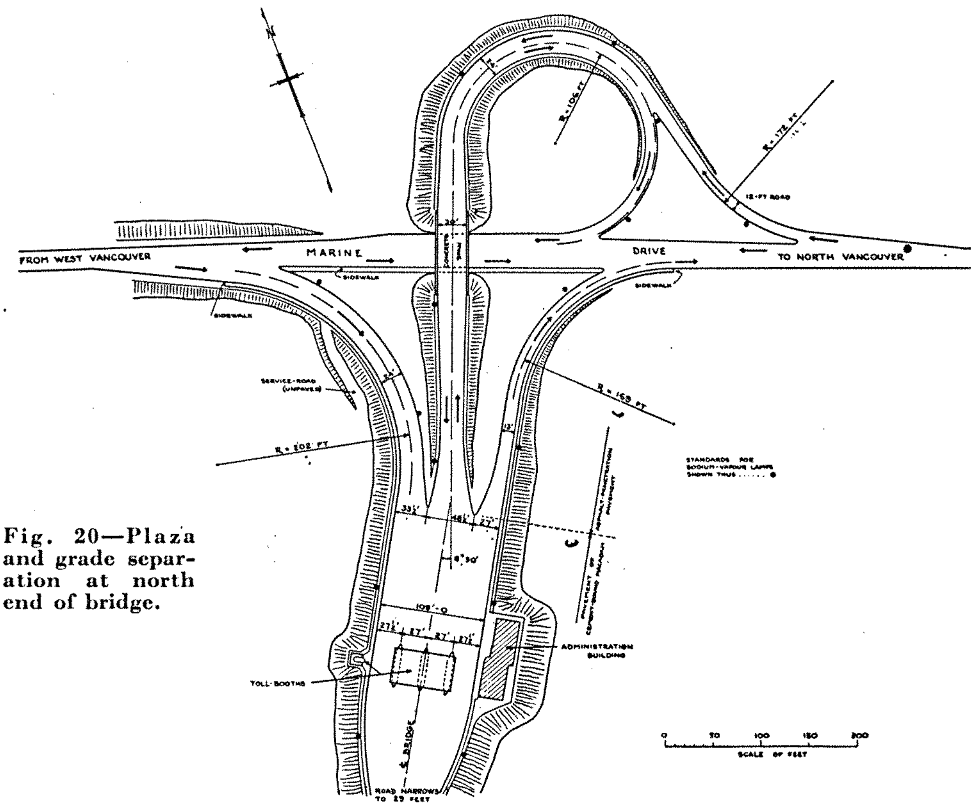

Fig. 20—Plaza and grade separation at north end of bridge.

For construction of the embankments, gravel fill of excellent quality was readily obtainable close at hand.

*This 4-in. parabolic crown has proved insufficient, the crest of the road being too flat for efficient drainage.

An extensive borrow pit was formed to the east of the right-of-way, on the Capilano Indian Reserve, suitable material being available after clearing the land and removing some 18 inches of top-soil. Although the original specifications contemplated a programme of heavy rolling to consolidate the fill as it was placed, it was found that, provided the gravel were spread in thin layers, consolidation was adequately secured by the heavy traffic incidental to construction. The gravel, which is naturally graded from a sandy soil to boulders of 8 or 9 in. in diameter, bedded down very rapidly and formed an extremely dense, solid and stable embankment, practically impervious to water. The side-slopes throughout were finished to a batter of 1½ to 1, and the berm-width varies from 2 to 3 ft.

The total amount of fill placed by the contractor was 110,400 cu. yd., of which 45,200 cu. yd. represented additional work involved in the Marine Drive grade separation. Consolidation of this latter fill (placed only a few weeks prior to the opening of the bridge) was expedited by jetting with water as it was dumped. The total unit cost, per cu. yd. in place, including preliminary clearing, did not exceed 30 cents.

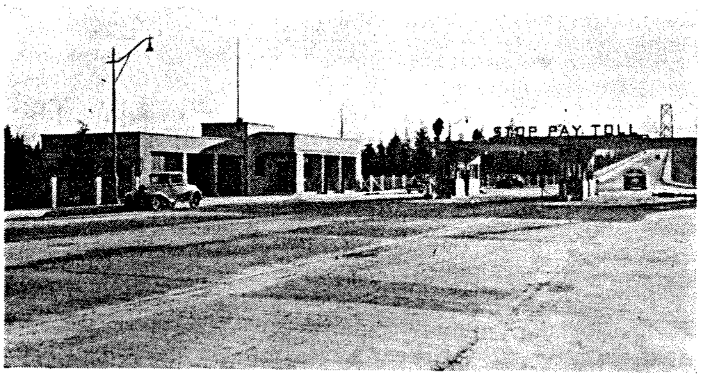

Photo Leonard Frank, A.R.P.S.

Fig. 21—North plaza, toll-booths, and administration building.

The original intention of the engineers had been to provide a temporary pavement on the embankment, to be replaced permanently after a suitable period of consolidation of the sub-grade by traffic and weathering. In view, however, of the remarkably solid nature of the fill, decision was made to proceed at once with the construction of a rigid slab over the main embankment from the end of steel as far as the division of the roadway. It was further decided to use the type of pavement known as "cement-bound macadam," since the prospects were that this could be placed more expeditiously and economically* than a regular concrete slab, while at the same time being of equal wearing-quality. An asphaltic pavement was, however, employed for the three roads of the Marine Drive junction.

Cement-bound macadam consists (to quote from the engineers' specifications**), of "a layer of coarse aggregate over which is poured a Portland cement grout of such fluidity that it will immediately flow through and completely fill the voids"; and the following is an outline of the method used in the construction.

Combined kerb-and-gutter-sections were first constructed of plain concrete, reinforced only by a "Truscon" kerb-bar, to serve as boundaries for the roadway, and the sidewalk slabs, 3½ in. deep and reinforced with a light mesh, were keyed into them. The subgrade was then levelled-off to profile, and compacted with a 3-ton roller. Expansion joints, both longitudinal and transverse (the latter at 30-ft. intervals), were provided for by one-in. planks laid on edge and extending from just below the subgrade surface to ½ in. below the finished grade. The coarse aggregate, a clean durable gravel screened to be principally of 2-in. size, was then spread evenly over the subgrade to the specified depth of 6 in., and lightly rolled to smooth the surface without damaging the stones. The grout mixture consised of 87½ lb. of cement to 175 lb. of clean natural sand of specified grading, mixed with sufficient water (about 7 gallons per sack) to obtain the required consistency, and its "fluidity" was defined by the time required for the mixture to flow from a vessel of specified size. The grout was poured over the stones by means of a wooden chute perforated with one-in. holes 3 in. apart, distribution proceeding continuously in order to avoid the formation of air-pockets. Thorough penetration was assured by constant supervision. Sufficient grout was applied to leave a thin film over the top of the stones.

The grout was then left undisturbed for one to two hours, and the pavement was compacted by rolling with a 3-ton roller until a hard even surface was obtained, hand-brooms being used to distribute the grout evenly and to remove any excess. Surface irregularities were removed by 12-ft. tamping templates, and excess water by dragging strips of damp burlap along the road. The final finish (Fig. 21) was obtained by transverse brushing with fibre brooms. The total area paved with cement-bound macadam was 1,730 sq. yd., the amount of stone used being approximately 800 cu. yd.

The embankment roadways are bounded by diamond-mesh wire fences 3 ft. high, carried on 8 by 8 in. cedar posts 9 ft. long. The posts extend 4 ft. 9 in. into the fill, being stabilized by underground cross-timbers: all embedded timber received two coats of preservative.

*Owing to the limited extent of the paved area, and also because of the contractor's lack of experience with this type of pavement, it is doubtful whether any saving was actually effected in time or money.

**Based on those of the Portland Cement Association.