Foundation of the Hampton Roads Tunnel

Fondations pour le Tunnel de Hampton Roads

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Summary

A description is given of the foundation problems of the subaqueous tunnel—a part of the Hampton Roads crossing. Foundation upon sand and silt. Silt excavation and sand backfill for the South Portal Island. Vibro-flotation compaction of sand fill for both islands. Checking of sand densities before and after compaction. Light weight fill.

The Hampton Roads Bridge-Tunnel crossing serves the vehicular traffic on the western side of Chesapeake Bay, eliminating the use of two ferry lines, one from Old Point Comfort on the north shore to Willoughby Spit on the south shore of Hampton Roads, and one from Newport News on the north shore to Pine Beach, Norfolk, on the south shore (Fig. 1).

For the crossing of the channel proper a subaqueous two-lane tunnel of the precast type (Thoresen, 1950; Steuerman and Foster, 1952) was designed.

The alignment of the tunnel was determined by the permit issued by the Corps of Engineers U.S. Army requiring a clear channel of not less than 3808 ft. width with a minimum depth from MLW to the top of the tunnel structure of 58 ft., within the limits of the prescribed channel. Compliance with the requirements of this permit brought the north portal of the tunnel into open water near Old Point Comfort and the south portal into open water near the old man-made island of Fort Wool. Trestles connect the tunnel with both shores. The total length of the water crossing is 18,364 ft. (3.48 miles) and the estimated cost $29,275,000.

The north portal location was considered satisfactory from the point of view of substrata, water depth, and protection against waves coming from Chesapeake Bay.

The required location of the south portal left much to be desired for proper foundation conditions since the substrata soils at this location consisted of an 80 ft. thick silt layer under 16 ft. of water. The existing Fort Wool island provides only a small degree of protection from waves coming from the direction of Chesapeake Bay. However, moving of the portal to the south (nearer to Willoughby Spit) where more suitable substrata exist would have meant not only lengthening the tunnel with consequent considerable increase in cost as compared with that of a trestle, but also greatly increased power requirements for tunnel ventilation. The length of tunnel developed as a result of fulfilling the requirements of the crossing permit was the maximum limit for ventilating it from two ventilation buildings, one located at each portal. The incorporation of a third or intermediate ventilating building was uneconomical and obviously impractical; increasing the cross-sectional dimensions of the tunnel for greater duct areas would have been expensive. Cost comparisons based on several schemes indicated that the shortest tunnel consistent with crossing permit requirements gave the most economical over-all solution and the portals were located accordingly.

Of the 43 test borings made at and near the centreline of the proposed crossing, two borings taken near the future portals reached a depth of 250 ft. below mean sea level; each of the remaining 41 borings were 150 ft. deep. Three additional...
boring near the south portal were taken to determine whether a change in the tunnel alignment would result in improved foundation conditions at this portal. These borings reached a depth of 100 ft., but the boring logs served only to prove the uniformity of the substrata in this general area.

In these boring programmes the number of blows on a standard sampler was recorded, undisturbed samples were taken from cohesive materials and analysed in the laboratory. The geological longitudinal profile (Fig. 2) shows that sand was found at the soil surface at the northern portal, and silt at the southern portal.

When a tunnel is supported on a sand stratum the question of the weight of the tunnel structure and of the backfill enclosing it is of small consequence since, practically speaking, any natural sand can support some additional weight without settlements of a magnitude sufficient to endanger the structure. On the other hand, when a structure subject to vibrating loads is placed on a loose, saturated sand there is a strong possibility of settlements occurring as a result of consolidation taking place within the stratum due to densification of the sand. In California, for example, hydraulic fills previously placed to depths of up to 50 ft. have had indicated settlements of upwards of 2 ft. under the effect of seismic disturbances. Structures founded on piles which were driven through these hydraulic fills to a penetration of from 10 to 20 ft. into compressible substrata settled as a result of the drag load effect on the piles.

At Hampton Roads seismic disturbances were discounted, but settlements caused by vibration from heavy traffic and rotating machinery (ventilating fans, etc.) were a distinct possibility and had to be considered. Such vibration could be of a magnitude and duration sufficient to cause compaction of loose saturated sands beneath and around the structure. The more the surrounding sands are compacted tending to act as a unit with the structure, the larger is the mass of the vibrated unit (structure-soil), consequently the more the vibrations are absorbed by this unit. Experience has shown that a 20-ft. layer of well compacted sand is sufficient to absorb the energy producing the vibrations and prevent the latter from being transmitted to the underlying strata. Vibrometer tests on the Elizabeth River tunnel, conducted two years after its opening to traffic, disclosed no oscillations readable by this instrument.

In the case of a cohesive type of substratum such as silt, the influence of weight and vibratory loads is opposite in effect to that upon sand. Energy-producing vibrations do not compact cohesive soils, but weight in the form of a surcharge does consolidate such soils. Hence for structures founded upon silt, the analysis of the influence of loads upon the consolidation of the substrata is of primary importance.

Each pre-cast tube section is designed to have a negative buoyancy of only 2 ton per linear foot which means that its weight in place in the tunnel trench is considerably less than the weight of the volume of silt displaced by the tube section. It was considered prudent, however, to protect the individual sunken tube sections and the finally completed underwater portion of the tunnel from damage caused by vessels or other floating plant that might sink on to it. The trench into which the individual sections are placed was dredged to such depth below natural bottom as to allow the placing of the section and permit a final cover of 5 ft. of backfilled sand without carrying this latter protection above the natural bay bottom. Where, however, the protection layer actually projects above the natural bottom, it was necessary to construct rock-fill gravel berms and blankets to hold this layer and prevent it from being scoured out (Figs. 3a and b).

As long as the tunnel protection remains below the natural ground surface, the total weight of tunnel plus the protection layer is smaller than the weight of displaced silt. If it were not for the fact that the weight of the sand backfill is greater than the displaced silt, the silt lying under the tunnel would tend to move upwards.

The heavy backfill has a tendency to compress the substrata under the tunnel. In order to eliminate consolidation of the substrata of the tunnel proper it is not necessary to have the weight of tunnel plus backfill equal to the weight of the displaced silt, but rather to balance out the forces acting (according to Boussinesq) upon the substrata under the tunnel such that the average force acting upon the soil at the edge of the tunnel and in the middle of the tunnel shall be zero, or shall be directed upwards. For the weight conditions at Hampton Roads it was determined that such a balance of forces exists so long as the protection layer does not extend above the natural bay bottom. Therefore, as shown in Fig. 2, a non-symmetrical longitudinal profile was adopted to insure that the tunnel was to the maximum extent possible at least 5 ft. below the natural bay bottom wherever the supporting substratum is a predominant silt.

At the southern change in tunnel gradient, the protection layer of sand backfill extends above the natural bay bottom (Fig. 2). Since at this point the tunnel structure is located at a great depth, only a relatively thin layer of pre-loaded silt will remain under the structure, and its consolidation will produce negligible, if any, settlements of the tunnel.

Although damage to the tunnel structures from ships is a remote hazard to be reckoned with, there is the always present danger of damage to the tunnel as well as to the trestle bridges and their foundations by collision from ships during storms and periods of extreme high water. However, the cost of repairing such damage to a trestle is small in comparison with the cost of repairing damage to a tunnel located very deep in the water. All parts of the tunnel lying above the natural bay bottom must have positive protection against damage by ships.

A study of different methods of protection indicated that artificial islands represented the safest and most economical means of protection for the transition structures from trestles

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**Fig. 2 Geological longitudinal profile**

profil en long géologique
Fig. 3 Tunnel sections—for location of A, B, C, D, see Fig. 2
Coupe du tunnel—pour emplacement des coupes A, B, C, D, voir Fig. 2
to the tunnel proper, and it was decided to build them at both portal locations.

The North Island was an ideal solution. Shallow water, sand substrata, and protection by Old Point Comfort made this type of construction easy. All that was required to obtain a stable island was to use good sand for the fill. Specifications for the sand to be used for the fill of both islands required:

'90—100 per cent shall pass a ¼ in. sieve and not less than 85 per cent shall be retained on a No. 200 mesh sieve. It shall be reasonably well graded between these limits in order to obtain the maximum possible relative density of placed material, shall contain not more than 3 per cent clay. . . .'

The South Island as heretofore stated represented a difficult construction problem. Laboratory tests on undisturbed samples taken from the 80 ft. thick layer of silt located at the site of the South Island showed that under the weight of this island, which would extend from —16 ft. MSL (existing bottom) to +11 ft. MSL (finished grade) Fig. 3D, a settlement of the magnitude of 5 ft. due to primary consolidation must be anticipated. Two ft. additional settlement would be expected from secondary consolidation.

The use of sand drains for consolidation of this silt was contemplated, but the time available for this consolidation could, at the very best, take care of the primary consolidation only. The danger of secondary consolidation of a magnitude detrimental to the tunnel excluded the use of sand drains to consolidate the substrata as a support for the tunnel.

Providing a pile foundation for the tunnel structure, within an island subject to settlement, was rejected after studies indicated the risks associated with maintaining such a structure and the excessive cost of construction.

After evaluating all of the above, the decision was made to design the South Island on the basis of excavating all of the silt from beneath the structures from the existing bay bottom down to its boundary with firm material (sand). The poor foundation material thus excavated was to be replaced with a hydraulically placed sand fill.

As a result of complete analyses of boring samples in the soils laboratory, the characteristics and properties of this silt were determined, allowing the prediction of the slopes on which this material would stand during excavation. These slopes were designated as payment lines for dredging and fill and corresponded very closely with the actual slopes developed.

For the foundation of the tunnel a high relative density of the sand fill had to be achieved. To have attempted to specify a sand of particular gradation with special techniques to place the same would have greatly increased the cost of this item considering that the quantity involved amounted to approximately 1,650,000 cu. yd. For this reason the decision was made to utilize sand hydraulically pumped from borrow areas on the bay bottom not far removed from the site. Every effort was made by the experienced contractor to place this material as stipulated by the Specifications as follows:

'It is also the intent . . . to obtain a fill of the maximum possible relative density. To this end the maximum distance from the bottom of the discharge pipe to the surface on which the fill is being deposited shall be 15 ft., unless otherwise permitted by the Engineer'.

The Contract Specifications required test borings from the surface of the future islands and compaction of the material by Vibroflostation (Steuerman, 1948; Mystkowski, 1953; D'Appolonia et al., 1955) wherever the density of the sand for a depth of 20 ft. below the future foundation was found to be less than that specified (63 per cent relative density).

Fig. 4 shows characteristic sieve curves of sands used for the fill of both portal islands. Grain sizes of each sand were quite uniform and the grains were well rounded. The sand for the South Island fill was coarser than that of the North Island fill and contained practically no grains passing the 200 mesh sieve.

Immediately after the completion of the sand fill on each island, test borings were made to check the density of the substrata for a depth of 20 ft. beneath the foundation slab of the structures. Uncased holes were specified and a Bentonitic slurry or 'drilling mud' was used to keep the bore hole open.

On the North Island, the first to be filled and tested, the procedure adopted was to depend mainly on the direct determination of the relative density of the undisturbed samples. Taking of undisturbed samples from the deep natural sand layers was made particularly difficult by the presence of a dense deposit of limonite concretions generally referred to as 'bog iron'. To effectively sample this layer the Denison type rotary core barrel was used, taking samples 6 in. in diameter and 12 in. long.

Because of a lack of correlation between undisturbed samples and the number of blows in a single bore hole, or between similar soil layers in adjacent bore holes, the method of securing undisturbed samples was subject to question. For the remainder of the testing it was decided to institute a complete parallel check by correlating the sampling with the number of blows counted in driving the standard sampler.

The sequence of operations normally employed thereafter was as follows:

(1) A drive sample was taken by dropping a 140 lb. weight 30 in. and counting the number of blows required to drive a 2-in. ID, split sampler each 6 in. increment of an 18 in. drive.
(2) The hole was drilled down an additional 18 in. and then cleaned out;

(3) a piston type sampler was employed to take an undisturbed sample approximately 2 ft. long.

The above steps were repeated in the same order for each 5-ft. section of the 20 ft. depth sampled.

The piston sampler consisted of a piston fixed rigidly to the drill stem, which in turn was clamped to the drilling rig located at the surface of the island, the latter utilized as a reaction. The sampler head, equipped with an attached Shelby sampling tube, slid on the piston rod being actuated by the hydraulic pressure of the rig's pump. An air bubble coming to the surface of the drilling slurry and a drop in the reading of the pump's pressure gauge indicated that the sampler had completed its stroke. Observations at the site showed that the 2 ft. stroke led to incomplete recovery, consequently the stroke was reduced to 18 and then finally to 12 in. Each time the sampler had been shot the drill rods and sampler assembly were completely withdrawn from the hole; during this period the bore hole was kept filled with drilling slurry. The sampler was very carefully lifted out of the slurry-filled hole and a cap placed over the cutting edge of the sampler tube while it was still several inches below the surface.

The relative densities indicated by these two methods—testing of undisturbed samples and counting of the number of spoon blows—proved to be incompatible when sampling in uncompacted fills. No correlation could be established between the relative densities obtained by these two methods either by comparison with the Burmister (1948) or the Bureau of Reclamation (1953, 1955) curves. In all cases the number of blows indicated a sand of lesser relative density than that found by the laboratory results on the testing of undisturbed samples (Fig. 5). Although the subject of much speculation, a positive determination of this discrepancy could not be found. It was presumed, based on sieve analysis, that the uniform well-rounded sand with no particles passing the 200-mesh sieve is susceptible to compaction under the influence of the driving action of the piston sampler. The number of blows recorded on the split sampler was so low that it was considered dangerous to assign structural vibratory loads to this loose water-saturated sand. It was decided, therefore, to compact both of the hydraulically filled sand islands for a depth of 20 ft. beneath the structures. Wherever the structures on the North Island were founded directly on natural dense sand, compaction of these areas was omitted.

The compaction pattern adopted for the use of Vibroflotation consisted of an equilateral triangle with sides of 8 ft.

The deeper compactions in the natural fine sands of the North Island required a longer than normal time since the fines reduced the permeability of this material, allowing the water to be squeezed out at only a very slow rate.

On the South Island the first compaction difficulty to be experienced was that of getting the Vibroflot through the upper layer of dry coarse sand having a high permeability. To overcome this difficulty an additional jet was used to force enough water into this upper layer to permit the lowering of the Vibroflot. After several trials with the latter, it was found more expedient to excavate the upper sand layer by bulldozing it to the side, and start the entry of the Vibroflot at this lowered level.

At the site of the Ventilation building the Vibroflot was required to penetrate 62 ft. into the island sand in order to compact the material 20 ft. below the structure, leaving the upper 42 ft. uncompacted since this latter material would be later excavated and replaced by the structure. This requirement was specified in order to finish all compaction work ahead of the trenching and excavating for the final structures, thus eliminating interferences and reducing the time required to keep the ground water level lowered.

To overcome the high resistance developed in the deep sand layer against the penetration of the Vibroflot, a ballast of 2 ton had to be attached to the upper end of the machine. Even with this additional weight the Vibroflot was able to penetrate to the required depth only so long as the sand was not pre-compacted by compaction in an adjacent triangular pattern. To avoid this pre-compaction, the normal 8 ft. triangular pattern was opened up to 12 ft.

The compaction of the coarse sand of the South Island, in contradistinction to the fine sand of the North Island, was accomplished relatively quickly once the technique of penetration

Fig. 5 Relative densities and number of blows on standard sampler before compaction (South Island)
Densité relative et nombre de coups sur carottier étalon avant compactage (Ile du Sud)
was developed. The checking of compaction results by comparing blows on the split spoon with laboratory results on the undisturbed samples was repeated in a manner similar to that used prior to Vibroflotation. The points for the determination

The correlation between these two methods for compacted sand is shown (Fig. 7) using the curves developed in the U.S. Bureau of Reclamation (1955), neglecting the theoretical 7 per cent increase in the number of blows upon the sampler used at Hampton Roads (2-in. ID) as compared with that upon the standard sampler (2-in. OD).

The curves relating the number of blows and the relative density were developed in the U.S. Bureau of Reclamation (1955) for pressure in lb./sq. in. in the soil at the elevation where the sample was taken.

At Hampton Roads the upper portion of the fill at both islands was 11 ft. above water; for this layer the weight of the compacted sand was approximately 100 lb./cu. ft. Therefore, at elevation zero the pressure was 1100 lb./sq. ft. = 7.5 lb./sq. in.

The buoyant weight of compacted sand is 60 lb./cu. ft., which means that every 6 ft. of buoyant soil corresponds to a pressure differential of 2.5 lb./sq. in.

The following pressures are computed:

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\begin{array}{c|c}
\text{Elevation} & \text{Pressure} \\
0 & 7.5 \text{ lb./sq. in.} \\
6 & 10.0 \text{ lb./sq. in.} \\
24 & 17.5 \text{ lb./sq. in.} \\
30 & 20.0 \text{ lb./sq. in.}
\end{array}
\]
In Fig. 7a results obtained on samples from compacted South Island fill taken between the elevations — 6 and — 30 are compared with curves from the U.S. Bureau of Reclamation (1955) for pressures equal to 10 and 20 lb./sq. in.; Fig. 7b shows samples from compacted North Island fill at elevation 0 to — 24 ft. with curves for pressures 7.5 and 17.5 lb./sq. in.

In addition to the test borings, it was a requirement of the Specifications that compaction results shall be further verified by the taking of undisturbed samples from subgrade after the trenches are fully excavated. A set of smallest relative density results obtained by surface sampling in a shallow test pit dug on the North Island is shown in Fig. 8.

The excavation of the silt from the area of the South Island introduced an additional problem, concerning the foundation of the tunnel tube lying at its northerly end. Here the back slope of the deep excavation resulted in one of the 300 ft. long tube sections being founded partly on the slope of the new sand fill and partly in the original silt of the bay bottom. It was recognized that the weight of the sand backfill in conjunction with the gravel berm and blanket protection layers could produce consolidation of the underlying silt and consequent non-uniform settlement of the tunnel. To overcome the unbalanced forces at this location, a light weight backfill of approximately 20,000 cu. yd. was specified for placing over this tube (Figs. 2 and 3c). The specification for the material forming the light weight fill required it to be an inert expanded shale or slate where the average submerged weight would not exceed 25 lb. per cu. ft. after removal of all fines floatable in sea water.

At the present time (May 1956) 15 out of 23 tubes forming the future subaqueous tunnel have been placed, both portal islands have been filled and compacted and the construction of the cast-in-place structures started on the North Island (Fig. 9). The cast-in-place structures will be built in dry trenches, the excavation being accomplished by lowering the ground water level by means of deep wells and well points.

It is expected that the Hampton Roads crossing will be opened for traffic in the late fall of 1957.

References


— and Foster, H. A. (1952). Elizabeth River and Baytown Tunnels. Civil Engineering, 1010

