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DEPARTMENT of TRANSPORTATION
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THE GEOLOGY AND CONSTRUCTION TECHNIQUES
OF THE SECOND HAMPTON ROADS CROSSING, NORFOLK, VIRGINIA

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The first Hampton Roads Bridge Tunnel between West Ocean View Street in Norfolk and the Hampton Shoreline near Old Point Comfort in Hampton was constructed during the period November, 1954 through November, 1957 (see Figure 1). The project was financed by part of a 95 million dollar bond issue and replaced the Newport News and Old Point Comfort ferries. The crossing consists of 23 tubes totaling 6,859 ft. in length, 2 man made islands, open approaches each approximately 600 feet in length and 2 approach trestles, the north trestle is 3,250 feet in length and the south is 6,150 feet. The elimination of tolls in 1974, due to retirement of the bonds indicated an increased traffic volume, consequently, the need for a second crossing.

The tunnel portion of the project consists of prefabricated tubes, cut and cover box sections, ventilation buildings and open approaches (see Figure 2). The 23 prefabricated tubes were sunk in place in a dredged trench and backfilled to provide a 5 foot minimum cover over the tunnel. At each end of the prefabricated tube section, box sections, which are each about 90 feet long and constructed by the cut and cover method, connect the prefabricated tube sections to the ventilation building structures. The open approaches carry the tunnel roadway to the trestles section.

The tunnel roadway descends on a 4.0 percent grade from a roadway elevation of +16.4 mean sea level at the north trestle to a low point of elevation -105.4 MSL in the tunnel, ascends on a grade of 0.5 percent under the main channel, and then continues on an ascending grade of 4.0 percent to a roadway elevation of +16.4 MSL at the south trestle (see Figure 3). The widths of roadway through the tunnel and open approaches is 23 feet between curbs and the minimum vertical clearance between the roadway and the ceiling in the tunnel is 14 feet.

Electric power for ventilation, drainage, and lighting is provided by submarine cables from each shore, and water for fire protection and cleaning is piped over each trestle. The roadway within the tunnel is a flexible bituminous concrete pavement on a reinforced concrete slab, and the interior finish consists of ceramic tiled walls and a porcelain enamelled aluminum pan ceiling.

The trestles both north and south are precast, pretressed concrete pile bents. The width of roadway is 30 feet between curbs, thus permitting traffic in either direction to bypass disabled vehicles. Two feet wide safety walks are provided on each side of the trestles and the concrete parapet walls are surmounted by aluminum posts and railings.

The second bridge tunnel is located inshore (west) of the first crossing, with parallel trestles joining enlarged portal islands (see Figure 4).

Hampton Roads is a large naval and commercial harbor in the Chesapeake Bay which falls in the Coastal Plain of Virginia. The Bay is about 10,000 years old. Its origin came during early Pleistocene times when the Atlantic Ocean advanced westward and drowned the valleys of the Susquehanna River. The tidal shore-line of the bay and its tributaries is estimated at 4,600 miles with a surface area of about 4,300 square miles. The waterways have a combined navigable length of approximately 1,800 miles.

Geologically, the Coastal Plain dips gently eastward at about 5°. It continues east beneath the Atlantic Ocean for about 200 miles where the continental shelf forms (see Figure 5).

The bedrock of the Coastal Plain is composed of partially consolidated detritus. The rocks consist of Cretaceous, Eocene and Miocene marine sands; commonly containing a large percentage of glauconite, gravels, clays, coquina and diatomaceous sediments. In turn, these sediments are sporadically overlain by Pleistocene and recent sand and gravel deposits.

The Paleocene period followed the Cretaceous. There is no evidence that indicates Paleocene sediments were deposited, which suggests a period of erosion or withdrawal of the sea. It has been argued that possibly the lower and older Eocene sediments may represent the Paleocene Period.

The Eocene began approximately 60 million years ago during another period of advancing seas. Depths of the sediments from the Eocene range from 150 feet in the west to 850 feet between Norfolk and the Cape Charles. The sediments are glauconitic sand beds with discontinuous interbedded clays. The formation abounds in fossil shark teeth and whale bones.

The absence of the Oligocene material in the
column indicates another era of either nondeposition or active erosion in recessed seas.

To the east, around Norfolk, the Miocene sediments reach thickness greater than 900 feet. The composition of the Miocene formation vary from sand to diatomaceous clays with some glauconitic clays and shell marls. The formation abounds in fossil remains of whales, seals, porpoises, mollusks, shark, ray and other fish.

The most recent system of rocks came during the Pleistocene Period. They may be simply described as unconsolidated, yellow cross bedded sand and gravel up to 100 feet thick near the Eastern Shore with a veneer of sand, silt and gravel overlying it.

The surface is only slightly dissected by streams. Swamps and marsh lands are common, with the Great Dismal Swamp being the largest. Because the coastline is so deeply cut by bays and estuaries, the appearance has been described as a "fringe of peninsulas". Bluffs up to 100 feet high commonly border the estuaries.

The stratification of the silt-clay and upper sand deposits was explored during the design of the first Hampton Roads Crossing by 46 borings (see Figure 6). Undisturbed samples were secured and tested to determine the soil properties. An additional program, consisting of 16 borings, was undertaken to further verify the soil profile. The later program substantiated the first. These indicated that the north 2,000 feet of the tunnel, portal island and trestle would be founded on sandy soils and would not present any special design or construction problems.

To the south, however, there is a thick layer of compressible organic silty clay which in turn is overlain by sand. The elevation at the bottom of the silty clay ranges from -30 feet at the south trestle to -130 at the deepest portion of the channel. Dense sandy 'soils' are found below the silty clay. The compressible material was a problem at the south Portal Island (see Figure 7).

The properties of the silty clay strata were investigated by means of field vane shear tests and laboratory tests on undisturbed samples, consisting of unconfined, undrained triaxial tests. A shear strength profile was developed from the data. Strengths varied from 300 psf, at the top to 800 psf at the bottom of the silty clay layer.

Compressibility of the silty clay was determined by consolidation tests in conjunction with evaluation of settlements observed at the final South Portal Island. The tests indicated that the silty clay layer was highly compressible and the required load would result in substantial settlements. Furthermore, the rate at which consolidation occurs is very slow; therefore, any design which is dependent on preloading would require some means of accelerating the consolidation.

As the construction of the new South Island would impose large additional loads on the compressible subsoils, then 3 schemes were studied to determine the method that would result in minimum cost and risk to the first island.

These schemes as discussed in the following sections are (1) the sheetpile trench method, (2) the undercut and backfill method, and (3) the sand drain and surcharge method (see Figure 8).

(1) With the sheetpile trench method of construction, the new tunnel and open approach structures would be located about 80 feet inshore (west) of the existing center line of the tunnel on the South Portal Island. Construction of the new facilities would be carried out within an open sheetpile trench parallel to the existing tunnel and within the confines of the existing dredged and filled island. This method would limit the need for extending the existing island and hence limit the new loads resulting from sand fill on the compressible substrata. However, the cofferdam methods requires the alignment on the new tunnel to be so close to the existing tunnel that the trench excavation would uncover the top of the existing tube. Therefore, any further consideration of this method was abandoned because of the high risk of disturbance.

(2) The construction of the original South Portal Island was accomplished by the dredging and disposal of 90 feet of compressible materials, and hydraulically backfilled with sand. Since completion of the construction 20 years ago, settlements have been within the predicted acceptable values, and have resulted in minimal corrective measures to the island and its protection, with no significant damage to the structures. Enlargement of the existing island by dredge and fill methods would require that the center lines of the existing and new island be 300 to 400 feet apart in order to avoid problems of slope stability during the dredging of the enlarged island. This method of construction, with suitable separation of the island, presented no undue hazards to the existing structures, but would require the removal and disposal of about 1,000,000 cubic yards of silts and replacement with approximately 1,500,000 cubic yards of hydraulic sand fill. Disposal at sea of unsuitable material is no longer economical and disposal in the Corps of Engineers Craney Island disposal area 5 miles distant requires a substantial rehandling charge. Further, the hydraulic fill to a depth of 20 feet below the new structures will require vibro-compaction for a distance of 25 feet each side of the tunnel center line. The total estimated cost is $5 million.
(3) Widening of the existing island utilizing the sand drain and surcharge method of construction would require placing hydraulic fill between the existing bottom (elevation -15 msl) and the top of the island (elevation +12 msl) within the enclosure of previously placed stone dikes. This procedure will limit silting and provide a dry land surface from which sand drains can be installed prior to surcharging to elevation +37 msl. After completion of the consolidation period, the surcharge would be removed and used as tunnel backfill. Settlement analyses were made using the Boussinesque Theory, and these were verified by the Finite Element Method using computer techniques. These studies indicate negligible additional settlement under the existing structures when the center lines between islands are 250 feet apart. This method would require 1,350,000 cubic yards of fill, 250,000 cubic yards of surcharge removal, 525,000 lin. feet of sand drains, and instrumentation. Estimated cost: $4.05 million.

Upon evaluation of the above factors it was decided to use the surcharge and sand drain method of construction. No dredging and disposal of unsuitable material is required in this method. However, the dredge and fill method is a positive and relatively foolproof plan while the sand drain construction and operation require considerable care and control of technique to produce completely satisfactory results. The South Island was constructed under a separate advanced contract to permit, under a subsequent contract, the trench to be dredged and deposited to construct the North Island and the tubes to be fabricated while the 12 months of surcharge on the South Island progressed.

The problems after the selection of the surcharge and sand drain method, became the criticality of design, care in installing the sand drains, and the evaluation of field instrument readings.

In general terms, the soil profile at the South Island site consists of a thin layer of loose silty sand at the bay bottom at about elevation -15. This is followed by a layer of clayey silt with seams and pockets of sand down to elevation -90. The lower portion of the silty clay contains some organic matter. Below the silty clay are layers of dense silty sand and sand (see Figure 9).

The clayey silt is slightly overly consolidated at the upper portion and normally consolidated lower down. It varies gradually in consistency from soft at the top to stiff at the bottom.

The presence of this clayey silt layer presents problems of stability and settlement.

Laboratory tests of undisturbed samples of the clay silt were made in 1954 for the first bridge-tunnel. Additional laboratory tests as well as field vane tests were made in 1969. The 1954 tests consisted of unconfined compression tests and triaxial tests. Only the 1954 unconfined compression test results were used for the design.

In the 1969 test 2 consolidated, undrained triaxial tests were made at chamber pressures equal to ½ the overburden pressure. These yielded results lower than those of the unconfined compression tests of the same sample tubes.

A plot of peak cohesion results versus depth showed that soil cohesion increased with depth. From this finding, plotting values of cohesion were assumed for design. The basis of the assumption was to follow the average lower limit of the test results, neglecting extreme values.

From a plot of Atterberg limits and natural water content versus elevation and stress-strain curves for laboratory and vane shear tests, it appeared that the clayey silt was quite sensitive, particularly in the upper layers.

Consolidation tests were performed during both the 1954 and 1969 investigations. Consolidation curves were produced by plotting preconsolidation pressure and existing overburden pressure versus depth and coefficient of consolidation, Cv, at applicable pressure ranges, versus elevation.

From these tests it appeared that:

1. The clayey silt is somewhat overly consolidated from its surface at -15 to elevation -50 and normally consolidated below that.
2. The time rate of consolidation is much higher for the soils above -50 than for those below.

For consolidation by means of sand drains, it was conservatively assumed that the horizontal coefficient of consolidation is equal to the vertical. Field permeability and pumping tests were performed during the initial stages of construction. It was planned that if it was confirmed that the horizontal coefficient of consolidation was substantially greater, the sand drain spacing would be revised. The sand drains were installed by jetting for minimum displacement, minimum smear.

On the basis of the relationship of the cohesion test results with overburden pressure and preconsolidation pressure, it was assumed that

\[ C = 0.3 \, p. \]

The initial cohesion is the product of existing preconsolidation, whether under existing overburden pressure or other geological preconsolidation pressure. When the soil consolidates under the
imposed load to a value higher than its initial cohesion, an increase in consolidation will take place.

The devices used to monitor the consolidation of the in situ foundation material were: (see Figure 10)

- Casagrande Piezometers
- Pneumatic Piezometers
- Deep Settlement Points
- Control Stakes
- Inclinometer Casings

Slope stability analyses were performed for various stages of the proposed construction. These consisted of complete solutions of the Swedish Slide Circle and manual solutions of sliding wedges.

The minimum factors of safety are tabulated below:

<table>
<thead>
<tr>
<th>Minimum F. S. which includes fill to the maximum elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slide Circle</td>
</tr>
<tr>
<td>+ 5</td>
</tr>
<tr>
<td>+12</td>
</tr>
<tr>
<td>+20</td>
</tr>
<tr>
<td>+30</td>
</tr>
<tr>
<td>+37</td>
</tr>
</tbody>
</table>

It should be noted that an approximate slope stability analysis by means of Taylor’s charts indicates that the fill to +12 requires the same average cohesion value as did the excavation for the existing South Island. It was reported that the excavation was made without any problems of instability.

For fills above elevation +12, the increase in cohesion due to consolidation of the sand-drained area was considered in the stability analysis.

The factors of safety tabulated above are considered adequate in view of the fact that the fill was kept under careful observation. It should be noted that the lowest factors of safety occur at early stages in the construction. For this reason, remedial action such as extending berms will be possible if the need is indicated.

Rather than introducing a mandatory waiting period of 3 months, which would interfere with the contractors’ schedule, the contract specifications provided that the engineer could direct suspension of all fill operations when the surcharge had been placed to the critical elevation of +20, should field observation indicate a need for additional consolidation time. A separate pay item for costs of delay time was provided. The decision whether to use the delay time depended on shear test results from borings drilled after the installation of the sand drains.

When the island fill reached elevation +12 piezometers were installed and read periodically. The sand drain installation proceeded using the spacing computed from low permeability values.

The new island consists of hydraulic fill confined within a rock dike and separated from it by a filter course. The rock dike with the filter course was placed on top of an underwater gravel blanket (see Figure 11).

In the past, an island of this nature would have been constructed by placing the hydraulic fill first and then protecting it with rip-rap. This procedure invariably pollutes the surrounding water, therefore, the dike was first placed around the entire perimeter of the new island and hydraulically filled inside. Color aerial photos were taken regularly during this operation, but no pollution was detected.

During the filling operation, periodic settlement readings were taken from settlement plates and deep points (see Figure 12). The effect of the sand drains were clearly distinguishable in the records with a total of 13 feet of final settlement (see Figure 13).

The island covered 23 acres with 17 acres in sand drains. Six thousand drains averaging 100 feet in depth, on 3 feet - 11 feet centers were installed.

Conferences with the U.S. Army Corps of Engineers, the U.S. Navy and U.S. Coast Guard established that adequate dredging spoil and borrow areas were available for the project. The U.S. Government has a disposal area known as Craney Island about 5 miles from the project that can accommodate pumped or bottom dump material. Approximately 100 acres of sand located just east of the South Island was established as suitable borrow.

The locations of leased oyster beds and public fishing areas in the vicinity of the project presented a problem to the dredging operation.
In addition to a survey of oyster bed ownership and yield, the Department requested the Virginia Institute of Marine Sciences (VIMS) to make a study of the shellfish population and to determine what effect the tunnel construction would have on these. VIMS made such a study before, during and after construction and reported that there was no noticeable effect on the bottom conditions (see Figure 14).

When the instrumentation indicated that the initial settlement has been achieved at the South Island, then the surcharge was removed by mixing the sand with water and pumping it across the bay with submerged lines to the North Island Site. The North Island had the same design except it had good foundation. A problem developed at the North Island when the cut and cover section was being made to transition the trestle into the tunnel. The island was hydraulically placed in such a fashion that any silt was pushed ahead of the filling operation. Unfortunately the island was so narrow that some of the silt was captured in the area of the transition and created a problem when de-watering for construction.

The photographs (following figures) illustrate the techniques by which the tubes for the tunnel were fabricated, filled and sunk into position.

Traffic counts indicate a 41% increase since the tolls were removed consequently the predications are reality.

The Department is making preliminary engineering studies for another similar crossing to Hampton Roads between Newport News and Norfolk, about 5 miles to the west of the present tunnels.

Acknowledgments

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FIGURE 9
CROSS SECTION (A-A) OF SOUTH ISLAND

FIGURE 12
The "outfitting" basin was located at the old coal piers in Norfolk harbor. The tubes, still afloat, were lined with an inner ring of concrete and fitted under a 23 foot roadway. This also included all utility and ventilation ducts. By the time all of the outfitting was done only 3 feet of the tube was above water.

The interior of the tube of the outfitting basin shows forms used for placement of the two lane roadway slab atop supporting haunches. Concrete was placed through a snorkel (center) via openings spaced along the top. Slat at lower right were for ventilation system's air ducts. When completely outfitted, a section weighed about 12,000 tons and worth about 2 million dollars.
The core of the tunnel consists of 23 double-shell steel tube sections, each about 350 feet long. The octagonal shaped outer shell of the tube measures 37 feet in diameter while the circular inner shell measures 33 feet in diameter. The sections were fabricated at Port Deposit, Maryland on special ways. The tube at right of photo is ready to be launched.

Enroute to the tunnel site, the completed steel tube has been fitted with watertight bulkheads at both ends and launched into the Susquehanna River which flows into the Chesapeake Bay. Fabrication of each section received approximately 600 tons of structural steel. About 1,500 tons of concrete was placed in the "Keel" between the inner and outer shells to serve as ballast.
After being outfitted, the almost completely submerged tube is maneuvered into position and lowered into a gravel bedded trench dredged across the bottom of Hampton Roads. To sink a section, additional concrete was pumped into pockets between the inner and outer shells. Once the tube became heavy enough to reach negative buoyancy it was lowered into position by the “lay down barge”.

Once in position on the bottom, the tubes were backed into position with hydraulic jacks and bolted and the remaining space between inner and outer shells filled with concrete. A solid ring of concrete was poured around each joint and as the steel bulkheads between sections were progressively cut through, the joints were welded from the inside and faced with an inner ring of concrete. Lying on the bottom in the trench the tube was covered with 5 feet of sand.