THE DESIGN AND CONSTRUCTION OF THE HAMPTON ROADS TUNNEL

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HISTORY OF THE SUBAQUEOUS TUNNEL

The Sicilian historian, Diodomus Siculus, tells us of the Babylonian Queen, Semiramis, who, for reasons best known to herself, wanted to be able to cross the Euphrates come hell or high water. She therefore ordered her Director of Public Works to build a tunnel under the river. This he did by diverting the water and building a tunnel with walls 20 bricks thick and a vaulted roof, sealed by a waterproof coating of hot asphalt. Unfortunately, no trace of this structure has been discovered so we have only the word of the ancient reporter that the Queen was at least twenty-four centuries ahead of her time, because, in contrast to all other types of tunnels, those passing under water are a product of relatively recent times.

Several unsuccessful attempts were made around the Year 1800 to tunnel under the Thames, but it needed the invention of the shield by Marc I. Brunel in England, to start the subaqueous tunnel on its way. Incidentally, he is supposed to have conceived the idea from observing one of the curses of waterfront construction: the cutter head of the Teredo Navalis.

The modern underwater tunnel may rightly be called a native of England, where several were built during the 19th Century, and improvements made in the design and construction of shields. One of these was the seven-foot diameter Tower Tunnel under the Thames, 1,350 feet long, first used for cable cars, later by pedestrians, and eventually for water lines. Its builder, James H. Greatheath, of South Africa, developed a shield incorporating all essential parts of the ones in use today.

In 1879, a small tunnel was built in Antwerp, Belgium, using for the first time a shield combined with air pressure, and in the same year this method was applied by D. C. Haskin, in the initial attempt to build a rapid transit tunnel under the Hudson River between New
York and Jersey City. It was only in 1905, on the third try, that the Hudson Tube Tunnels were finally completed. Two years earlier, between 1901 and 1903, the East Boston Tunnel had been built entirely of mass concrete, without cast iron or steel rings, using an elliptical shield.

While several early vehicular tunnels were built during the nineteenth century, the main demand for underwater crossings came from railroads, to which were added, towards the turn of the century, the needs of the rapid transit systems, newly developed in such cities as New York, London and Paris. Beginning around 1920 a new wave of activity in subaqueous tunnels was created by the automobile, and this has not subsided yet, as attested by the many projects completed in recent years.

**Shield Driven Tunnels**

About one-half of the eighteen modern vehicular tunnels, and most of the rapid transit tunnels, have been built by the shield method. The largest concentration of these is undoubtedly in the New York area, with its extensive network of railroads, subways and highways, and its many waterways. The Sumner Tunnel in Boston also belongs to this family.

From a shaft sunk on shore or from a deep open cut, a shield is advanced by pressure from a series of hydraulic jacks. This shield consists of a steel face, usually circular in shape, and of a diameter slightly larger than that of exterior dimension of the tunnel structure. The soil ahead of the shield is removed through ports, protected by bulkhead type doors, either by letting it flow in, if it is sufficiently loose, as the shield advances, or by digging and cutting, if more solid. From the face a cylindrical skirt extends back a short distance to form a working chamber within which to erect the tunnel lining rings. In order to keep water from entering this chamber, it is filled with compressed air with sufficient pressure to balance the maximum hydrostatic head. An adequate thickness of overburden of sufficient density is required above the shield to prevent this compressed air from blowing out at the top. If necessary a clay blanket several feet thick may have to be deposited on the river bottom. Men and materials are moved through air locks built into the tunnel a short distance behind the shield. As the latter advances, rings made of cast iron or welded steel segments are erected and bolted together. These rings are usually about 30 inches wide, and they form the struc-
HAMPTON ROADS TUNNEL

The interior of the tunnel is lined and finished to suit its function, be it for railroad or highway use.

SUNKEN TUBE TUNNELS

Since shield driven tunnels are expensive, particularly in labor costs, and advance slowly, other construction methods were searched for at an early stage. The idea of building tunnel sections on land or in basins and sinking them in place is nearly as old as the shield. Boston has one of the earliest trench tunnels, namely: the 9-foot diameter brick and concrete sewer tunnel built in the outer portion of the Harbor in 1893-94, sunk in 52-foot sections.

Figure No. 1 shows the cross sections of a number of tunnels...
built for railroads, rapid transit or vehicular traffic by the sunken tube method. A combination steel shell and reinforced concrete section with an octagonal exterior shape, which, with some modifications has become somewhat of a prototype for many later tunnels, was first used for the sunken tube portion of the Detroit-Windsor Tunnel. This tunnel is the only one for which both sunken tube and shield construction was used, the former for the river portion, the latter for the land sections, built under city streets.

One of the most recent sunken tube type tunnels is the Hampton Roads Crossing.

**HAMPTON ROADS TUNNEL**

(a) *General Description:*

Water is the life blood of the historic Hampton Roads area, whence ships have sallied forth for peaceful purposes, and for war, ever since the birth of this nation, but it also forms major obstacles everywhere to the unhindered movement of land traffic.

Preliminary studies for a fixed crossing of Hampton Roads to replace two ferry routes started over twenty years ago, finally culminating in the project completed by the Department of Toll Facilities of the Virginia Highway Department last November. The 3,700-foot width of the channel and the importance of this entrance to the Norfolk Naval Base precluded a bridge, both from financial, as well as strategic considerations.

Figure No. 2 shows a plan of the ultimate project which begins as a dual, limited access highway at Route 168 in the north, extending through Warwick and Hampton, and crossing Hampton Roads with bridge and tunnel structures between the Phoebus Shore and Willoughby Spit in Norfolk, where improved Ocean Avenue and a new dual roadway lead into Tidewater Drive, a main arterial highway into downtown Norfolk. Of the approximately 22½ miles of final project length, the main crossing covers close to 3½ miles. Of these all but 4,000 feet are over shallow water, varying from a few feet to 25 feet in depth. The main channel measures 3,700 feet between the 6-fathom lines, with at least 58 feet of water required over the tunnel backfill for a distance of 3,170 feet. The maximum natural depth of the channel reaches close to 70 feet.

These circumstances obviously suggested a tunnel under the main channel, connected to the shores by trestle-type bridges and cause-
Fig. 2.—Plan of Hampton Roads Project.
ways. Consideration had been given in earlier studies to building the tunnel between Fort Monroe and Fort Wool Island, which would have eliminated the north trestle approach and reduced the size of the new south island. However, the restricted military area at Fort Monroe and the fact that Fort Wool had been constructed mostly of stone fill dumped into the silty bottom, which would have posed a tricky problem, eliminated this solution from further consideration.

In order to obtain the optimum length of tunnel, both from the standpoint of general economy and ability to provide ventilation from buildings located near the ends, without prohibitive power demands or enlarged cross-section for increased air duct space, its portals had to be located as close to the channel lines as possible. After lengthy studies, which will be referred to later, the termination of the tunnel in two artificial islands seemed to best fulfill the various requirements. Figure No. 3 represents a plan and profile of the tunnel crossing.

This tunnel has a length of 7,479 feet between portals, of which 6,860 feet are made up of 23 sunken tube sections, each about 300 feet long. The remaining 310 feet on each end, which also include the foundations for the ventilation buildings, were built by the cut-and-cover methods in open trenches. Open approaches roughly 600 feet long on each island bring the roadway up to elevation +12 above mean tide.

Figure No. 4 shows three typical cross-sections of the completed structure; namely, the U-shaped section of the open approaches, the reinforced concrete section of the cut-and-cover portion, and the composition structural steel and concrete body of the sunken tubes.

As illustrated in Figure No. 5, the basis of the sunken tube structure is a cylindrical shell, 33 feet in diameter, made of 5/16" steel plate. Longitudinal stiffeners, of 5" x 3/8" plates or 4" Tees, as required by loads, are welded to the outside of these shells between transverse external stiffening diaphragms spaced 14'-10", which have an octagonal periphery. The steel cylinder forms a watertight membrane and all joints, which were welded throughout, were carefully tested for even the minutest leaks. In order to contain the exterior concrete envelope of the tube, ¼" form plates are attached to the diaphragms and supported from the shell by struts and stiffeners. These steel tubes are fabricated and assembled by methods similar to those used in ship construction. For the Hampton Roads Tunnel, they were built in the Eddystone Plant of the Baldwin-Lima Hamilton
FIG. 3.—GENERAL PLAN AND PROFILE.
Corporation. Subassemblies 14'-10" long were built in the shop. In an assembly yard along the river, these rings were welded together into complete tubes. Since the narrow water and the relatively high bank made free launching impractical, the tubes were skidded onto a launching platform carried by triangular, wheeled carriages, running on rails supported on ground ways. The movement of this platform was controlled by winches. Before the ends of the tubes were closed

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**Fig. 4.—Typical Tunnel Cross Sections.**
by steel bulkheads, prior to launching, the reinforcing steel for the inside concrete lining was placed, and the keel part of the outside concrete envelope was poured. Once the initial production difficulties were overcome, the 23 tubes were produced at the rate of one every three weeks.

In order to conform to the changes in the roadway grade, two of the tubes were provided with mitred joints.

**Fig. 5.—Cross Section of Sunken Tube.**
The completed tubes were towed through the Delaware-Chesapeake Canal and through Chesapeake Bay to an outfitting pier at Lamberts Point, some five miles from the tunnel location.

While floating at this pier, the 18-inch interior reinforced concrete lining on the roadway slab was poured, access being available through a number of hatches in the top. After completion of the interior concrete, the hatches were closed by welded steel plates and the top cap of the exterior concrete was poured. With this the tubes had a freeboard of two feet and were ready for towing to the site.

(b) Design Analysis of Tubes

From the moment of launching to the time they rest in their final position and are covered by backfill, the tubes go through a number of stages of different loading and stress conditions.

*Design of Steel Tubes:* Stresses during the control side-launching are nominal. After the tube is launched and floating, its weight is evenly distributed, except for the heavily braced end bulkheads, which cause a bogging moment, producing a longitudinal bending stress of about 9,000 psi in the steel shell and stiffener ribs. In order to keep the longitudinal moments on the tube as a whole to a minimum, the placing of the interior concrete, which is done in several lifts, is started near the quarter points and proceeds symmetrically in both directions. On the other hand, the exterior water pressure produces both transverse and longitudinal bending stresses in the shell plate and bending moments in the diaphragm frames as the draft of the tube increases, due to the continuously added weight of the concrete lining. For the critical design conditions of the steel tube, immersion to the horizontal center line has been assumed, with shell plate and diaphragms fixed at the upper edge of the keel concrete, or about 45 degrees below the horizontal. The opposing action of the freshly placed lower lifts of the concrete lining has been neglected. These assumptions are somewhat conservative since the rate of concreting generally does not produce quite such large increments of submersion. The stresses in the various parts have been analyzed as follows:

*Transverse Bending in Shell Plate:* The shell is figured as a continuous plate supported by the longitudinal stiffeners and subject to exterior pressure equal to the hydrostatic head. The maximum corresponding bending stress in a flat plate would be 30,000 psi, which,
however, is reduced due to the additional stiffness produced by the curvature.

**Longitudinal Bending:** The shell plate and stiffeners acting jointly, transfer the pressure to the diaphragm frames, whereby they are undergoing longitudinal bending stresses amounting to 16,000 psi.

**Diaphragm Stresses:** The diaphragm frames were analyzed under the assumption that they were fixed at the top of the keel concrete and cut at the top, where tangential force $H_c$, shear $Y_c$ and moment $M_c$ were applied to hold the equilibrium. The moment at any point $D$ located at an angle $U$ from the top is:

$$M_u = m_u - H_c R (1 - crs U) + M_c$$

where $m$ is the moment of the external forces.

$R$ is the radius of the shell.

From the continuity of the arch, equations are derived which permit the calculation of the numerical values of $H_c$ and $M_c$, the shear $Y_c$ being zero due to the symmetry of the structure and loading. The maximum stresses in the frame occur at the top of the keel concrete, they are 12,700 psi in tension and 9,100 psi in compression.

*Design of Composite Tube*

In the completed tube the reinforced concrete lining and the steel parts act as a composite structure, subject to two distinct loading conditions; namely water pressure when fully submerged during the sinking operation, to which is added the load of the backfill after the tube is in place in the trench.

If a uniform cylinder of circular cross section has a weight just sufficient to cause full submergence, it will be subject to direct stresses only but not to bending. While the tunnel section does not quite conform to this condition, it was considered as a sufficiently accurate approximation to justify the assumption that during the sinking no bending stresses did occur in the tube cross section.

The loads imposed on the tube by the backfill may be subdivided as follows:

- Vertical load above top of tube, uniformly distributed over horizontal diameter.
- Horizontal pressure due to uniform vertical load, uniformly distributed over vertical diameter.
- Triangular horizontal pressure due to earth between top and bottom of tube.
Vertical load from earth in exterior quadrants between top of tube and horizontal axis.

Vertical reaction of the tube on the foundation, uniformly distributed over the horizontal axis.

Bending moments and direct stresses for these various loads were computed by means of formulas developed by James M. Paris (2).

For calculating the stresses in the tubes from the above loads the active structure was assumed to consist of the interior reinforced concrete shell, 18 inches thick, the steel tube and stiffening frames, and the keel portion and the top cap of the exterior concrete which were poured in the dry. The rest of the outside concrete, being poured by tremie, was considered to be ballast only. Based on analysis of soil samples, the following weights of materials were assumed for the design of the sunken tube sections:

- Weight of moist earth, in air, 120 lbs./ft.³
- Weight of submerged earth, in air, 70 lbs./ft.
- Hydrostatic coefficient, 0.27.
- Maximum unit stresses under critical design loads are:
  - 20,000 lbs./in.² tension in structural steel.
  - 18,000 lbs./in.² tension in reinforcing steel.
  - 1,380 lbs./in.² compression in concrete.

A minimum 28-day compressive strength of 3,000 psi was specified for the structural concrete of the tubes.

(c) **Sinking of Tubes**

After completion of the concreting of the tubes at the fitting-out pier they are towed into position over the previously dredged trench and enough ballast concrete is added to the outside pockets to overcome buoyancy. The section is supported by floating derricks or special sinking barges which control its lowering into the trench.

Sighting masts temporarily attached to each end of the tube serve to control the alignment.

In order to set the tubes to proper grade, a sand foundation course at least two feet thick has been placed in the bottom of the previously dredged trench, and levelled to exact alignment by a heavy steel screed, suspended from a carriage supported on barges by rails set parallel to the grade.
To form the joints between tubes, steel collars of the same diameter as the circular shell have been welded to the end bulkheads, from which they project a distance of four feet. At one end of the tube a 36” hood plate is attached to the lower half of the collar; on the opposite end a similar hood is attached to the upper half of the collar. These hoods protrude for half of their width from the collars, the lower one being attached to a tube already in place, forming a cradle for the collar of the next tube. To allow for easy fitting, their inside radius is an inch larger than that of the collars. After a tube has been sunk, it is drawn into contact with the previous one by means of ratchets operated by divers in order to minimize an accumulation of error in aggregate length. Five-inch diameter tapered pins are then inserted into matching steel castings riveted to the collars to hold the tubes together. The vertical edges of the square end bulkheads are quipped with sections of sheet pile interlocks welded to them. Into these are inserted curved steel plates carrying matching interlocks. The space around the tube joints thus enclosed by these plates and the foundation course is then filled with tremie concrete. This seals the joints sufficiently so that it can later be drained from the interior and the bulkheads can be cut out. A closure plate is then welded to inside stiffeners attached to the collars on each side of the joint, thereby completing the continuous, watertight steel skin of the tunnel. The tubes are bedded in special sand fill up to their centerline, and covered with ordinary backfill to a minimum depth of five feet over their top. The tunnel is then ready for the finishing operations of filling in the gaps in the concrete lining and roadway slab at the joints, erection of the ceiling and installation of tiling, electrical and mechanical equipment, and last the placing of a four-inch wearing surface of bitumastic concrete.

Sinking of the tubes was started at the north island, proceeding through Section No. 7, then sections No. 23 and 22 were placed at the south island. Sinking was then resumed from the north. This sequence permitted work on the islands to progress while the remainder of the tubes were sunk. On the average, a tube was sunk every three to four weeks. Special provisions were made at the joint between Sections 21 and 22 to allow for an accumulated error in length, but fabrication, sinking and survey had been so accurate that this adjustment amounted only to a few inches.
(d) Soil Conditions

Borings made at 500-foot intervals along the center line of the project encountered two different materials, as indicated on the soils profile (Figure No. 6). The area of the north island and the adjacent side of the channel are free of silt, and firm sand bottom prevailed throughout. Under the channel, which is up to 70 feet deep and self-cleaning, was found a layer of silt about 30' thick, partly covered by 10 to 30 feet of sand. Under the region of the southern island, and for a strip about 1,500 feet wide along the south side of the channel, the layer of silt reaches to a depth of 80 to 90 feet below the bottom of the water, which here is 17' to 20' deep.

Since the final weight of the tunnel is only two tons per lin. foot, the combined weight of tunnel and backfill produces a balance of forces exerted upon the sub-stratum along the edges of the tunnel and under the middle which is zero or directed upwards, as long as the top of the backfill does not project above the natural bay bottom. No settlement will therefore take place in the up to 20 feet of silt still remaining in part of the trench after it had been dredged. In order to reduce to a minimum the length over which the backfill had to be raised above the bottom, the low point of the tunnel profile was shifted north from the midpoint. In these areas the fill was extended to 100 feet on either side of the center line and protected against scour with rock-filled gravel berms and blankets.

A special problem arose on the south island where Tube 22 was founded partly on the original silt and partly on the sand fill of the island. In order to prevent settlement, the backfill over this tube consists of expanded shale weighing not more than 25 lbs. per cu. ft. when submerged in sea water.

(e) Island Construction

The north island, resting on firm sand, presented no problem except that of ultimate compaction of the fill to a density sufficient to support the superimposed loads without settlement. Fill material meeting the requirements that 90 to 100 per cent pass a $\frac{3}{8}''$ sieve and not less than 85 per cent be retained on the 200 mesh sieve, containing not more than 3 per cent clay, was dredged from the new Hampton Creek Channel, relocation of which was necessitated by the project.

The deep silt on the south side, on the other hand, did pose a
problem of considerable magnitude. Off-set borings indicated that the condition prevailed over such a distance that it could not be avoided by a shift in the center line. Several solutions were studied; namely:

Supporting the cut-and-cover tunnel, ventilation building, and open approaches on piles, without any island fill, in which case it would have been necessary to protect the structures against damage from any vessel straying from the channel, during storms or high water, by a ring of cellular cofferdams.

Supporting the structures mentioned are piles driven through a sand island, whose main function would be their protection against ships. Continued settlement of the fill would absorb as much as 75 per cent of the capacity of the piles due to load transferred by friction, and would present a continuous maintenance problem.

Consolidation of the silt by sand drains was ruled out by the loss in time.

Dredging out of the silt and replacing it with a sand fill was finally selected as the most efficient solution. Laboratory tests on undisturbed samples indicated that excavation slopes of 2:1 at depth below elevation —50' and 2.5 ∶ 1 in the upper layers could be achieved without difficulties. Sand of satisfactory quality, with practically 100% retention on a 200 mesh sieve was available within easy pumping range of a hydraulic dredge.

(f) Consolidation of Sand Fill

During the filling operations, the bottom of the discharge pipe was maintained at all times within 5 feet of the surface on which the sand was deposited, in order to obtain as dense a fill as possible. However, this was not sufficient to produce a minimum of 63 per cent relative density considered necessary for a depth of 20 feet below any structure to adequately support it without piles. A series of borings were made after completion of the fills from which undisturbed samples were taken to verify these conditions. The specifications provided that under these circumstances the fill had to be consolidated by Vibroflotation. On the north island, compaction was not necessary where structures were founded directly on the natural dense sand. For the fill, a consolidation pattern of equilateral triangles with 8-ft. sides was adopted. The fines contained in the material reduced its permeability to an extent that compaction time had to
be increased beyond that normally required in order to squeeze out the water.

Compaction took place before excavating for the final structures. Under the ventilation building on the South Island, only the layer between 42 feet and 62 feet below the surface was consolidated, since the material above it had to be excavated anyway. Difficulties were experienced in getting the Vibraflot to penetrate the upper layer of coarse sand, which was therefore removed by bulldozer. For the deep compaction, an extra ballast of two tons was added to the machine and the pattern was opened from 8 ft. to 12 ft. to enable the Vibraflot to penetrate to the required depth. Results achieved by this compaction were most satisfactory, as borings and undisturbed samples taken from subgrade after excavation indicated.

(g) **Storm Protection**

The north island is sheltered against severe storms by the projection of Old Point Comfort and by Fort Wool.

A three-foot thickness of well graded rip-rap, with maximum size rocks of about 2,000 lbs. placed on a 1½-foot gravel bedding course, sloped 1 \( \div \) 4 from MSL to \(-5.0\) and 1:2½ above MSL to elevation \(+7.0\) has given adequate protection.

The east and south sides of the south island, however, are exposed to heavy northeast storms with a reach of over twenty miles, making them subject to severe wave action. This danger becomes particularly acute during hurricanes combined with extremely high tides.

A very heavy rip-rap built up of granite quarry stones of up to 10 tons in weight was therefore decided upon to protect these sides (Figure No. 7). This rip-rap has an average thickness of 4 ft. resting on an 18-inch bedding course of quarry tailings on a uniform slope of 1:4. Its toe rests against a rock dike which extends from a base at elevation \(-12.0\) to elevation \(-4.0\). The heavy rock protection is chinked with quarry tailings similar to those used in the bedding. A seawall made of three tiers of concrete monoliths built up to an elevation of \(+20.0\) feet, further protects this side of the island against overtopping by wave action. In order to prevent damaging tidal currents to build up between the south island and nearby Fort Wool, the gap was closed with a rock protected dike. The west and north
FIG. 7.—SOUTH ISLAND PROTECTION.
side of the island are much less exposed and rip-rap similar to that of the north island was considered adequate.

During the years 1954 and 1955, the Norfolk area was visited by several severe hurricanes. The frequency and fury of these storms led to intensified studies of these phenomena by the U. S. Weather Bureau and other agencies and as a consequence, a great deal of new information became available.

The top of the islands is at elevation +11, the roadway grade at the end of the open approaches at +12, with an additional three feet protection afforded by the solid parapet around the tunnel approaches. These were ample safeguards against flooding of the tunnel under the highest storm tide on record. It seemed prudent, however, to take advantage of this new hurricane data to make a new appraisal of the most critical conditions which might be produced by a combination of the worst possible factors, such as extreme spring tides, combined

AERIAL VIEW OF SOUTH ISLAND.
with a near stationary hurricane centered over the area. This study established the fact that tide levels as high as $+13$ with 16-foot waves could happen, which would threaten to flood the tunnel subjecting it to excessive settlement and severe damage. This led to the decision to install tide-gates inside each tunnel portal, which will keep flood waters out of the tunnel. Flooding of the open approaches would not be too serious, and they could be pumped out in a short time. Closing of the tunnel in such an emergency would be immaterial, as all approach highways and city streets would be under several feet of water, stopping all traffic movements in the area. Since electric power would undoubtedly be lost for the duration of such a storm, auxiliary diesel engine-generators in the ventilation buildings would supply power to operate the drainage pumps and the lights necessary for maintenance personnel.

(h) **Tunnel Ventilation**

Under maximum conditions the tunnel requires a supply of about 1,700,000 cubic feet of fresh air per minute. This is supplied through the duct space below the roadway slab and flue outlets spaced about 15 feet apart along both sides, discharging the fresh air above the curbs. An equal amount of vitiated air is exhausted through ceiling ports into the upper duct.

Each ventilation building contains four supply and four exhaust fans of 213,000 cfm maximum capacity each. The use of axial flow fans instead of the traditional centrifugal fans, which have been customary in tunnel ventilation, permitted a considerable reduction in the size of the buildings. Adjustable pitch blades and two-speed motors, built into the propeller hubs, permit adjustment of air quantities to suit the traffic demands. Both speed and pitch are remotely controlled from the main control board located in the south ventilation building.

(i) **Drainage and Fire Protection**

Fresh water is supplied to the tunnel from both ends through mains connected to the city systems on the shores. In case of fire, booster pumps in the ventilation buildings will raise the pressure to 125 psi.

Interceptor catch drainage from the open approaches and carry it to sumps inside the portals from where it is removed by auto-
matically controlled pumps. A sump at the low point collects water from tunnel washing or fire-fighting inside the tunnel, which is then pumped into the nearest portal sump.

In order to reduce the propagation of fire in the total through burning fuel in case of a collision, a closed drainage system with catch basins is used instead of the open gutter installed in most other tunnels.

**Electrical System:** Electric power is furnished to the tunnel from each end by high tension feeders, originating in completely independent parts of the public utility system, thus insuring permanence of supply. Each of these feeders is capable of carrying the entire maximum power load.

Continuous fluorescent lighting provides good visibility in the tunnel, with adjustable high intensity of 35 foot candles at the portals, tapering off to 7 foot candles in the interior.

(j) **Tunnel Ceiling**

Instead of a concrete slab, faced with ceramic tile, the tunnel ceiling consists of porcelain enamel finished aluminum panels attached to steel stringers, which are suspended from the concrete lining by monel hangers. The panels are filled with two inches of concrete for added weight and to prevent vibration. The reduced thickness of the ceiling gives some additional air duct space and the panels are easily replaceable in case of damage. This construction also eliminates the hazard of fragments of tile and concrete being propelled from the ceiling during a severe fire.

At the request of the designing engineers, the Aluminum Company of America collaborated in testing the aluminum panels in their laboratories to check their behavior under high temperatures to which they may be exposed in fires such as have occurred in tunnels. The results were satisfactory and confirmed the decision to select this material.

**Conclusion**

In conclusion, the question may be asked: where can the sunken tube tunnel be used? The main requirements are the following:

(a) A reasonably stable bottom in which a trench can be kept open without undue difficulty for a sufficient time to prepare the foundation course and sink the tubes.

(b) Conditions of the shores so that the sunken tube trench
can be extended sufficiently far to permit the maintenance of a dike to protect the open trench for the cut-and-cover sections. If adjacent properties do not allow the trench for the tubes to be excavated with open slopes, a braced trench may be practicable for a short distance, into which a tube can be entered submarine fashion under the bracing, as was done in the Elizabeth River Tunnel in Norfolk.

Wherever these conditions prevail, a sunken tube tunnel can be constructed at a great saving in cost compared with that of a shield driven tunnel.

**REFERENCES**