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Geotechnical Aspects of the Fort McHenry Tunnel - Design and Construction

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ABSTRACT

The geotechnical considerations affecting the design and construction of an immersed-tube tunnel are presented. Construction of the tunnel required deep excavations in overconsolidated fissured and slickensided clay deposits. Undrained, long-term residual and fully softened drained strength parameters were determined. The depth of excavation and the life of the cut slope were considered in selecting the design parameters for the slope study analysis. A tie-back soldier pile and lagging system was used for protection of the east end excavation in a congested urban area. Each tie-back was tested according to a simple acceptance criteria developed during construction. Lateral displacements measured behind the excavations were negligible. Hydraulic dredging of the trench was accomplished with the aid of cutterheads. Dredge spoil was pumped into a contained offshore disposal facility. Flocculants were added to the dredge slurry to accelerate settling of the solids before the effluent was discharged into the Harbor. Construction of the immersed-tube tunnel was completed on schedule.

INTRODUCTION

Fort McHenry Tunnel, is presently under construction in Baltimore's congested harbor. The eight-lane $1 billion project is the widest vehicular tunnel ever built by the immersed tube method.

The tunnel comprises twin binocular fabricated steel-lined concrete sections. The binocular sections, each 92-foot-6-inch wide, are separated transversely by 10 feet and are placed in a common dredge trench 5,400-foot long. The immersed tube tunnel section terminates at a ventilation building at each end, with cut-and-cover sections extending both east and west for a portal to portal length of 7,200 feet (Figure 1).

The major construction contract (immersed tube section) consisted of dredging of a trench, placement of 32 binocular tube sections, relocation of utilities including a 48-inch water main, relocation of railroads at the east and west approaches, demolition of industrial buildings and temporary relocation and reconstruction of berthing facilities. It also included construction of an offshore contained dredge disposal facility for 3.5 million cubic yards of dredge material.

Unlike the west side, where the tunnel sections were placed practically at the harbor's edge, the tunnel on the east side coming under the 48-foot deep navigation channel had to extend about 1200 feet landward to the east ventilation building.

The substructure of the ventilation buildings and west and east approaches beyond the tube...
section are being constructed in dewatered excavations using the cut-and-cover method. In the cut-and-cover tunnels, northbound and southbound traffic lanes are in separate concrete box structures.

This paper presents the geotechnical aspects of the design and construction of the immersed tube tunnel.

SITE GEOLOGY AND SOIL STRATIGRAPHY

The site lies within the Atlantic Coastal Plain Geologic Province. The major coastal plain deposits are the Patapsco and Arundel Formations of the Potomac Group of the Cretaceous age (Ref. 1,2,3). These formations are overlain by Pleistocene to Recent Alluvial deposits and Manmade Fills.

The geologic and stratigraphic section is presented in Figure 2.

SUBSURFACE INVESTIGATIONS AND LABORATORY TESTING

Twenty three standard 3-inch diameter borings were drilled and undisturbed samples were taken along the tunnel alignment. In-situ field vane shear tests were performed in soft estuarine deposits. Thirty nine large diameter borings (4-inch and 5-inch) were drilled to obtain large undisturbed samples (3-inch and 4.5-inch) in the Cretaceous clays for comparative testing. Denison double tube core barrel soil sampler was used in the large diameter boreholes to obtain 2-3/8 inch diameter undisturbed samples of the hard Cretaceous clays.

In order to determine the frequency, orientation, and inclination of joints, fissures and slickenside planes in the overconsolidated Cretaceous clays, continuous sampling, with orientation carefully monitored, took place in two boreholes at the end where major excavations were to be made. The structural discontinuities were observed to exist over a wide range of inclination varying from subhorizontal to near vertical. Although the lateral and vertical extent of the discontinuities could not be determined from the borings, it was believed that the planar defects were randomly oriented and not continuous over significant lengths. The variegated Patapsco clays exhibited a blocky structure with fissures probably more tightly distributed than the upper gray Arundel clays.

Laboratory testing included determination of Atterberg limits, grain size distribution, moisture content, density, specific gravity, organic content, shear strength, consolidation and swell characteristics.

To determine the design shear strength of the overconsolidated fissured clays, the undisturbed samples were tested as follows:

a. Unconfined compression tests (AASHTO designation T208) and Unconsolidated Undrained Triaxial (UU) tests (AASHTO designation T234) were performed on specimens with diameters of 1.4, 2.8 and 4 inches. The UU tests were performed at the confining pressures representative of the specimens' in-situ pressures.

b. Istropically Consolidated Undrained Triaxial (CU) tests with Pore Pressure Measurement (AASHTO designation T234) were performed on soil specimens of 1.4 and 2.8-inch diameter. The majority of

The surface soils in both the east and west land areas consist of miscellaneous sand and clay fills placed during the development of the Baltimore Harbor. Underwater, the surface soils are Recent Harbor Bottom deposits consisting of dark gray clayey silt with decomposed organics and traces of petroleum products. Within the areas of the flowing waters of the Patapsco River the harbor bottom deposits are underlain by Estuarine silts containing traces of organics.

The Arundel formation is composed of dark gray to brown silty clay containing abundant siderite concretions. Some thin sand lenses are encountered in the clay beds (Ref. 2,3).

The Patapsco Formation consists of thick beds of clay interbedded with lenticular, cross-bedded sands. Because of the lenticular nature of the sediments, the deposits grade laterally from sand to clay. The claybeds are massive to crudely stratified, gray brown and variegated red in color (Ref. 2,3).

Various zones of indurated materials interspersed at irregular intervals were encountered within the Cretaceous clay and sand strata. These zones consist of siderite, siltstone and conglomerate layers of thickness varying between a few inches to a few feet.

During late Tertiary and Quaternary ages, hundreds of feet of Potomac Group sediments were eroded away. In addition, tectonic deformation took place along the fall line, a few miles from the project site (Ref. 7). The tectonic movement and erosional unloading caused joints and shear zones or fissures to form within the deposits.

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specimens were consolidated to their in-situ effective overburden pressure. Two specimens were consolidated to pressures equal to twice and four times that of the overburden. In all cases, side drains were provided to facilitate equilization of the pore pressure within the specimen as loading conditions changed. Specimens were consolidated with 30 pounds per square inch (psi) or more back pressure. The time rate of consolidation was monitored and used to determine the appropriate strain rate for each test. Deviator stress was applied only after the 'B' coefficient was greater than 0.95. The duration of the CIU tests ranged from one to three days.

c. One series of three Isotropically Consolidated Drained Triaxial (CID) test (AASHTO designation T234) was performed on 2.8-inch specimens. These tests followed a procedure similar to that of the CIU tests except that the applied back pressure against which the specimens were permitted to drain was 20 psi. The average CID test duration was 5 days.

d. Seven sets of three Consolidated Drained Direct Shear (CDDS) tests (AASHTO designation T236) were performed on 2.5-inch diameter specimens. The first specimen tested in each set was consolidated to its in-situ overburden pressure. The consolidating pressures of subsequent tests were twice that of the preceding test of the set. The strain rates were determined from the coefficient of consolidation for each specimen. Each test was taken to at least 20% strain and three tests were taken to 40% strain. The higher strains were obtained by returning the shear device to its original position after 20% strain had been achieved.

DESIGN PARAMETERS

The properties of the various soil types as obtained from the laboratory test results and the design values used are presented in Table 1. Selection of undrained and drained shear strength parameters of the overconsolidated clays for design of deep trench excavation slopes are discussed below.

Overconsolidated Cretaceous clays contained structural discontinuities of varying intensity and inclinations. The shear strength along these discontinuities is much less than that of the intact material, as discontinuities constitute planes of weakness (Ref. 7,8,9). As a result, the strengths measured in the laboratory had a wide scatter depending on the size of specimen, spacing, frequency and inclination of discontinuities and failure along any preexisting plane of weakness.

The highest undrained strength of 4600 pounds per square feet (psf) was obtained in intact specimens free from any weak plane and the lowest undrained strength (700 psf) was obtained when a specimen failed entirely on a preexisting weak plane of fissure or slickenside (Figure 3). Although in several cases 4-inch specimens yielded lower strength than 1.4-inch specimens (Figure 3), the results of undrained strength tests on
specimens of 1.4-inch, 2.8-inch and 4-inch diameter did not indicate any apparent trend within this size range. This may be due to the limited number of tests and erratic distribution of fissures and joints in the test specimens.

Figure 3 presents the peak drained shear strength as measured in the CID tests. Drained parameter of C'=2600 psf, φ'=14° are indicated from this plot. The peak drained parameters measured from the direct shear (CDDS) tests were C'=0 to 1400 psf, φ'

Figure 3. UU Triaxial Test Results—Arundel Clays

Table 1. Soil Properties

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>W (%)</th>
<th>γt (pcf)</th>
<th>OCR</th>
<th>Undrained Strength Measured in the Laboratory</th>
<th>Design Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granular Landfill</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>115</td>
<td></td>
<td>CIU CID Drained</td>
<td>Undrained Drained</td>
</tr>
<tr>
<td>Clay Fills (CL, CL-ML)</td>
<td>43±5</td>
<td>22±5</td>
<td>21±4</td>
<td>28±5</td>
<td>115</td>
<td></td>
<td>550-1500 500-850</td>
<td>c'=0 φ'=27°-30°</td>
</tr>
<tr>
<td>Harbor Bottom Deposits and Estuarine Silt (CH)</td>
<td>83±9</td>
<td>47±8</td>
<td>37±9</td>
<td>125±30</td>
<td>85</td>
<td></td>
<td>50-850</td>
<td>700 c'=0 φ'=23°</td>
</tr>
<tr>
<td>Over Consolidated Grey Arundel Clay (CL, CH)</td>
<td>47±10</td>
<td>23±4</td>
<td>24±6</td>
<td>20±5</td>
<td>125-135</td>
<td>4-8</td>
<td>700-4600 c'=0 φ'=29 c'=2600 psf φ'=14°</td>
<td>2000 c'=600 psf φ'=21°</td>
</tr>
<tr>
<td>Over Consolidated Variegated Potomac Clays (CH, CL)</td>
<td>52±12</td>
<td>22±4</td>
<td>30±10</td>
<td>19±3</td>
<td>125-135</td>
<td>4-8</td>
<td>600-6000</td>
<td>2500 c'=600 psf φ'=21°</td>
</tr>
</tbody>
</table>

Experience indicates that fissures or discontinuities open up during and after excavation due to stress relief. With time and stress, the fissures or discontinuities gradually tend to elongate into more continuous cracks. Thus, the time the excavation slope remains open and the access of free water to the slope have important effects on the shear strength characteristics of the clay. The access of free water helps to soften and lubricate the weak planes of discontinuity reducing the shear strength to a residual value along those planes. Therefore, the undrained shear strength is unconservative for long term stability (Ref. 5,7,10).

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= 18°-26° (Figure 5). The above peak drained parameters, however, are not representative of the long-term strength of the soil after excavation.

In the slow drained test, the tested specimen exhibits a peak strength then falls significantly to a residual value with continued strain (Ref. 5,6,7,10,11). The reduction in the shear strength is a function of continued dilation and smoothing of the failure surface. The long-term drained strength on a fissure or slickenside plane can be represented by the residual value measured (θ'~r = 14°-20°).

The selection of drained shear strength parameters for the slope stability analysis was based on the following considerations:

a. Since the excavated slope will not remain open for a long period of time (3 years maximum), the shear strength of the clay may not reach the long-term residual value.

b. Since no pre-failure plane was apparent in the clay specimens, the average drained strength can be represented by the normally consolidated peak (C'=0, θ'n.o. peak) identified as the fully softened strength of the soil (Ref. 11).

c. At the base of the deep cut slopes where the overburden pressure remains significant, the fissures or joints would probably be tightly closed and would not allow excessive softening, thus retaining cohesion during the life of the open trench excavation.

d. Application of the fully softened shear strength in terms of θ' only (C'=0) would make the slope stability analysis independent of the depth of excavation and is generally used for permanent slopes. For the relatively short life of the trench cut, a small cohesion intercept (c') along with fully softened θ' would provide relatively steeper but economical design.

Based on the above considerations, the drained strength parameters of C'=600 psf and θ'=21° were subjectively selected and used for design of the trench slope under drained conditions (Figure 5).

**EXCAVATION SLOPES**

The major portion of the wet trench excavation is within the limits of the harbor where slopes are essentially submerged. The design slopes for the submerged excavation were 1 vertical on 1.5 horizontal in Creataceous silty sands and silty clays, 1 vertical on 2.5 horizontal in relatively soft estuarine silts, and 1 vertical on 4 horizontal in very soft harbor bottom deposits. A 10-foot wide berm was generally provided at the interface between the more competent Creataceous clay stratum and the relatively softer estuarine silt and harbor bottom deposits. The berms provided an increased factor of safety and served to collect material sloughing from the softer soils promoting a relatively clean excavation bottom. A typical trench slope is shown in Figure 6.

The west end of the wet trench excavation was partially submerged and cut through a significant depth of landfill material. The design slope in this area was 1 vertical on 2 horizontal.

The excavated slopes at the east end were rather deep, with depth of slopes ranging between 65 ft and 95 ft. The design slope at

**Figure 4. CID Triaxial Test Result**

**Figure 5. Drained Direct Shear Test Result**

**Figure 6. Excavation Slope at Harbor Channel**
this location was 1 vertical on 1.5 horizontal in the Cretaceous soils which were generally submerged. Excavation of the upper soils was protected by a support system necessitated by the congested use of the area.

Simplified Bishop slip circle, and sliding wedge analyses were performed for determining the stability of the slopes under short term and long term conditions utilizing the soil parameters described earlier. The drained analysis yielded lower safety factors. A minimum acceptable safety factor of 1.2 was used for design.

EAST END PROTECTED EXCAVATION

At the east end of its alignment, the construction of the 8-lane vehicular tunnel required a trench 179-foot wide at the bottom and over 400-foot wide at the top with the depth of excavation ranging between 65 feet and 95 feet from the ground surface (Figure 7).

Such a wide excavation through a busy port facility required careful engineering and economic considerations. In the interest of economy, and to minimize disruption of existing port activities, it was decided to excavate the trench slopes as steep as possible and to combine the open cut with an excavation support system (Figure 8).

Various types of earth retaining structures were investigated. The two most economically feasible support systems consisting of a steel sheet pile bulkhead or a soldier pile and lagging system were specified. The selection of the system was left to the contractor. Because of the width of the excavation and the presence of an underwater berm at the slope, bracing support systems were not feasible. Instead, tiebacks were specified with a maximum allowable load of 35 tons. The maximum allowable tieback spacing was 10 feet, vertical and horizontal, in most areas and 8 feet in critical areas. The contractor installed a tied-back soldier pile and lagging system.

The specifications required three tieback anchor tests for each type of soil encountered in the anchor zone for development of tieback bond length. These tests required application of 200% of the design load in 20% increments at one-hour intervals except for the final load increment which was to be retained for 24 hours. Axial strains were to be recorded at 0, 1, 2, 3, 4, 5, 7, 10, 15, 20, 25, 30, 45, and 60 minutes after each load increment. Additional readings were required at 1, 2, 4, 8, 16, and 24 hours for the final load. Axial strains were to be plotted versus load increments to be used for determination of the allowable loads for the tiebacks. The allowable loads also had to meet the creep criteria requiring:

![Figure 7. East End Excavation Plan](image-url)
that creep curves plotted in log of time should not indicate accelerated movement with time.

In addition, each production anchor was to be proof loaded to 140% of the design load in 1 ton increments at one minute intervals. The proof load had to be maintained for 30 minutes. A loss of 15% or more load in 30 minutes would mean rejection of the tieback.

The method of constructing the anchor, the tendon type, method of corrosion protection and the bond length was not specified so that the contractor may provide an economical tieback system while satisfying the design and specification requirements. The geotechnical report (Ref. 4) which was a reference document in the bid package recommended that tiebacks be anchored in the competent Cretaceous soils not in the fills.

Straight shaft anchors consisting of 1-inch Dywidag bars were installed inside 3-inch diameter holes in sandy soil and 6-inch holes in soft clays. The tieback subcontractor performed 11 tieback anchor tests and installed 92 production tiebacks prior to submission of the test results and design bond lengths for approval. Upon review of the installation documents it was found that a large number of tiebacks were installed in the variable sand and clay fills and that only 4 out of 11 tests performed satisfactorily. Based on the four satisfactory tests, the contractor developed a design bond length of 15 to 20 feet for all production anchors.

Because a large number of production anchors were installed and because any delay would result in a substantial time loss in construction schedule, it was decided to evaluate the actual capacity of each tieback already installed. Wide variations in anchor performance with respect to capacity and creep movement were observed. Because of the variety of soil types (fill) existing in the anchor zone, it became evident that the specified proof test criteria developed for competent Cretaceous soils should not be used for tiebacks in fill. Instead, a simple acceptance criteria was developed for each production tieback as follows:

1. Proof loads were to be applied in increments not to exceed 25% of the 140% proof load except that the last load increment must not be less than 15% of the proof load. Creep movements were to be monitored with an independently supported dial gage capable of reading to 0.001 inches. Gage readings were to be recorded at 5 and 15 minutes after application of the final load with the jacks adjusted, if required, to keep the load constant. The tiebacks would be considered acceptable if the creep movement between 5 and 15 minutes was less than 0.027 inches.

2. Unacceptable anchors would be retested at a reduced load. Retesting would continue until a load level meets the acceptance criteria. The acceptable design load would be calculated by dividing the acceptable load level by 1.4

3. If an anchor failed to carry the full proof load, the proposed design load would be reduced to 50% of the maximum load carried by the anchor. The anchor would then be proof tested as indicated above to confirm the adequacy of the revised capacity.

Based on the above criteria, the majority of the already installed tiebacks were accepted many at reduced capacity and additional tiebacks were installed to supplement the required capacity. The new tiebacks were installed with 20 to 30 feet of bond length. Most of the new tiebacks were accepted at their full design capacity. The tiebacks performed satisfactorily for the 2-3 year duration of the project. Figure 9 is a photograph of the east-end excavation area after installation of the support system.
INSTRUMENTATION

A field instrumentation program was implemented during construction to monitor ground movements at the excavated slopes, the excavation support system, and existing structures; and to permit timely implementation of proper remedial measures when and as required. The instrumentation program included topographic surveys of vertical and horizontal movements, and the reading of piezometers and inclinometers. The average measured lateral displacements in the soil behind the excavation were of the order of one half to one inch.

DREDGING

Dredging of 3.5 million cubic yards of material down to 110 feet below MLW was accomplished by a 27-inch hydraulic dredge (Figure 10). Different cutter heads were used for dredging the varied in-situ materials. Significant resistance was encountered in dredging the Cretaceous soils requiring frequent repair and replacement of the cutterhead teeth. The approximate average rates of dredging for soft and hard materials were 20,000 cubic yards and 4,000 cubic yards per day respectively. Figure 11 shows two different cutter heads used for dredging of soft and hard overconsolidated clays.

A dual phase excavation was performed at the east end. Land excavation methods were used to remove materials from the surface to Elevation 0.0 leaving a soil plug at the channel. Below that level, hydraulic dredging was used.

BACKFILLING

A screed barge followed the path of the dredge, screeding about 2 feet of gravel foundation course on the floor of the trench to receive the tubes. A screed plow, 63 feet wide, was suspended from a bridge crane and used to screed the foundation (Figure 12).

After the tunnel elements were placed and connected, a granular backfill was placed.
along both sides of the tube up to the spring line, using tremie pipes, to lock the tubes in place. The granular fill was then placed to a minimum of 6 feet above the tube.

On land, backfill material above H+1.0 was compacted to allow future construction and operation of port facilities.

DREDGE DISPOSAL

Construction of the tunnel involved dredging of 3.5 million cubic yards of Baltimore Harbor material. After evaluating several upland and harbor disposal sites, a 146-acre site in the Baltimore Harbor, about 2 miles from the tunnel, was selected as the only location that provided sufficient capacity and met both design criteria and environmental requirements regarding dredging, disposal, and construction.

The $60 million disposal facility, which included a 5,600 linear-foot-cellular cofferdam containment structure and 20-foot high clay-lined perimeter dikes was constructed so that it could be converted into a port terminal at a later date (Figure 13).

The disposal facility was designed to receive 3.5 million cubic yards of dredged material consisting of 600,000 cubic yards of highly contaminated, very soft, harbor bottom deposits, 650,000 cubic yards of organic clayey and sandy silt deposits, 1,200,000 cubic yards of gravelly sands, and 950,000 cubic yards of stiff-to-hard clays and clayey silts. Different bulking factors were used to predict the increased volume of these materials after being dredged and redeposited. The increased volumes of different materials and the rates of deposition into the facility, sedimentation of the solids, and effluent discharge into the harbor were used to estimate the required size of the disposal facility.

The principal environmental requirement governing the design of the facility was that the discharge from the facility should not contain more than 400 parts per million of suspended solids. Since the size of the site was not large enough to allow adequate settling, flocculants were added to the dredged slurry to accelerate settling of the solids before the effluent could be discharged back into the Baltimore Harbor. The hydraulically dredged material was pumped into the contained facility and the effluent was diverted through shaft-type weirs into treatment and settling basins before discharge. The flocculant used in this project was Calgon's Polymer M-502.

The successful construction and operation of the dredge disposal facility was instrumental in the successful construction of the immersed tunnel on schedule.

CONCLUSIONS

Geotechnical considerations influenced to a great extent the design and construction of the immersed-tube tunnel. Excavation of the trench, backfilling, disposal of dredge spoil, and excavation support at the east end were important factors affecting the schedule and construction cost of the project.

The overconsolidated Cretaceous clays existing at the site contain fissures and structural discontinuities of varying intensity and inclination. The undrained shear strength of the clay is inherently statistical in nature, depending on the size of specimen, and spacing, frequency and inclination of the discontinuities. A shear strength value of 2000 psf was selected for design under undrained conditions.

Peak, long-term residual, and fully softened drained strength parameters were determined. Shear strength parameters of C' = 600 psi and φ = 21° were selected for design of the trench cut slopes under drained conditions. The depth of excavation, and the temporary nature of the cut were considered in the selection process.

Simplified Bishop slip circle and sliding wedge analyses were performed for determining the stability of the slopes under short and long term conditions. The drained analysis yielded lower safety factors. The excavation at the east-end of the tunnel was protected by a tied-back soldier pile and lagging system. A maximum tie-back load of 35 tons was allowed. Each production tie-back was tested according to a simple acceptance
criteria developed during construction of the excavation support system. Lateral displacements measured in the soil behind the excavation were of the order of one half to one inch.

Dredging was accomplished by a 27-inch hydraulic dredge with different cutter heads. The average dredging rates for soft and hard materials were 20,000 cubic yards and 4,000 cubic yards per day respectively. 3.5 million cubic yards of dredge spoil were pumped into a dredge disposal facility with a containment structure consisting of 65-foot-diameter cells. Flocculants (Polymer M-502 from Calgon) were added to the dredge slurry to accelerate settling of the solids before the effluent is discharged back into the harbor. The discharge water contained less than 400 parts per million of suspended solids.

The immersed-tube section of the Fort McHenry tunnel and its dredge disposal facility were constructed on schedule. No major problems were encountered during construction of the tunnel.

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LIST OF REFERENCES


6. Chandler, R.J. and Skempton, A.W., "The design of permanent cutting slopes in Fissured Clays" Geotechnical 24, No. 4, 1974


8. Lo, K.Y "The Operational Strength of Fissured Clays" Geotechnique 20, No. 1, 1970

