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Rebuild of US 27

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ABSTRACT

An existing four lane section of US 27 was widened to eight lanes. The project lies in the rugged Valley and Ridge Province of the Appalachian mountains starting at the Tennessee River in Chattanooga, Tennessee and extending 2.6 km north to Signal Mountain Boulevard. This terrain presented many challenges to accommodate the road widening in the narrow, rugged topography. Multiple tiered wall systems consisting of large tieback walls, post-tensioned reaction blocks, and gravity MSE walls for a total of approximately 30 retaining walls were required. The retaining walls served to stabilize the thrust faulted geology while also minimizing right-of-way acquisitions. The cut walls consist mainly of a tiered system of tied-back soldier piles walls with cast-in-place concrete facing. Cast-in-place pile shear walls were required along the toe of existing MSE walls to maintain stability during construction. This paper discusses the geotechnical assessments for design and construction, slope stability analysis, landslide mitigation alternatives, soldier pile wall design features, site challenges, monitoring and performance (to date) of the wall systems.

INTRODUCTION

This project involves the proposed widening of US Highway 27 (State Route 29) from the north end of the Tennessee River Bridge at Manufacturer's Road to State Route 8 (Signal Mountain Road), approximately 2.64 km of roadway (Fig. 1). The project site is located adjacent to the Tennessee River on the north side of downtown Chattanooga, Tennessee in Hamilton County. The existing, limited access highway is to be widened from four lanes (two northbound and two southbound) to a minimum of eight lanes. Some portions of the alignment will expand to as much as ten lanes where northbound and southbound ramp acceleration/deceleration lanes coincide.

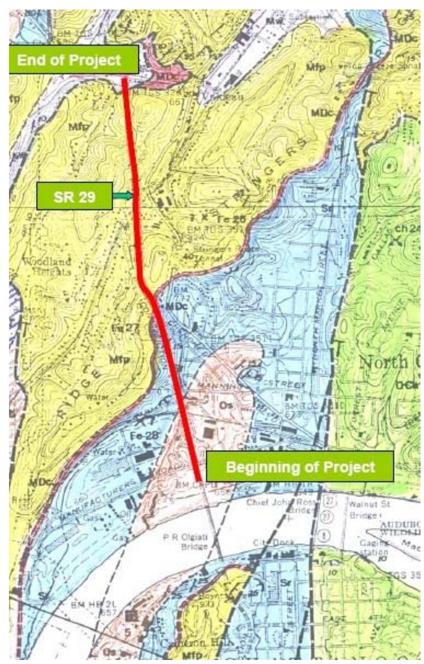

Fig. 1. General Project Alignment

In addition to the number and heights of cuts and fills involved, a major factor in determining an economical design for the alignment is the complicated geologic conditions along the roadway alignment. The existing roadway is situated between several steep cut slopes, some of which are only marginally stable or currently exhibit signs of instability. Due to the proximity of the project to the downtown area, the steep slopes along the alignment, and the relatively dense population near the alignment, widening the pavement and constructing cut and fill slopes to a stable configuration would result in major land acquisition for slope right-of-way. Accordingly, retaining walls were selected as a means for providing roadway support and reducing cut volume, acid producing waste, and right-of-way acquisition.

SITE INVESTIGATION

A limited amount of subsurface data for the project was available from a preliminary Tennessee Department of Transportation (TDOT) exploration program along with some data that the geotechnical engineer had developed on adjacent, commercial sites. A site reconnaissance was performed and an extensive drilling program initiated. The steep topography required the use of a skid rig and bulldozer along with conventional truck-mounted drill rigs. A total of 239 additional borings were drilled, of which about 2,400 m was in soil and 475 m was core-drilled in rock. Over 370 samples were subjected to laboratory tests including 16 CU triaxial tests with pore pressure measurement.

GEOLOGIC SETTING

The project site is located in southeastern Tennessee. The region is part of the foothills of the Appalachian Mountains and the geologic setting of the site is the western portion of the Valley and Ridge Physiographic Province. Hamilton County spans the boundary between two of the major physiographic provinces of Tennessee, the Cumberland Plateau to the west and the Valley and Ridge province to the east (Fig. 2). The Cumberland Escarpment, a nearly 245 m high bluff that marks the easternmost edge of the intact Cumberland Plateau, is located approximately 3.2 km west of the project. The project site is along the westernmost ridges (Stringers Ridge and Laurel Ridge) of the Valley and Ridge province in this area. Elongated ridges and intervening valleys, all trending in a northeast-southwest direction, characterize the physiographic province.

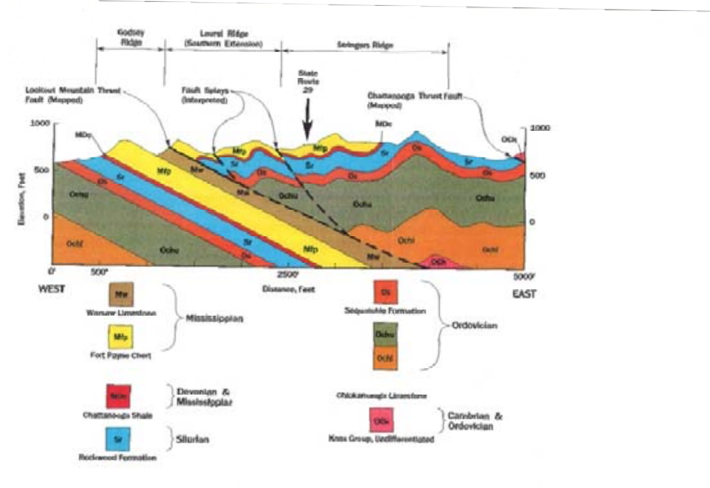

Fig. 2. Generalized Geologic Section

As a general rule, the Valley and Ridge province is underlain by sedimentary bedrock, including limestone, that has been folded and faulted, resulting in steeply dipping beds. Differential erosion has created ridges where rock types are more resistant to weathering and valleys where "weaker" or more soluble rock types outcrop.

The project alignment is trending slightly west of north, crossing the axis of northeast-southwest trending Stringers Ridge and, in the north portion of the alignment, a dissected portion of Laurel Ridge. Between those ridges and the Cumberland Escarpment (3.2 km west) is a broad valley that is underlain by the Lookout Mountain anticline. The eastern flank of the anticline that underlies the project area dips downward to the east at about ten to thirty degrees. However, within the project area, there are major thrust faults that outcrop on the east and west sides of Stringers Ridge and greatly complicate the geologic structure. According to published literature and the USGS Geologic Quadrangle (Finlayson et at, 1964), the bedrock that makes up Stringers Ridge and Laurel Ridge has been folded, faulted and transported several kilometers westward from its original position. Therefore, the fault west of the project site indicates the juncture between the underlying, uniformly dipping bedrock of the Lookout Mountain anticline and the faulted, fractured, and complex geologic structure encountered. In addition, there are other faults, mapped and unmapped, throughout the area.

The structural complexity of the project geology was created by multiple episodes of folding, fracturing and faulting. In particular, the northern portion of the project alignment has both major and minor through faults, many unmapped, which result in overturned blocks, mixtures of bedrock types where fault movements have jumbled various bedrock types and significantly weathered zones extending many tens or hundreds of meters beyond actual faults.

Published geologic maps of the area indicate that at least five bedrock formations and two significant (but inactive) thrust faults occur within the project limits. The borings confirm that the mapped formations are present. However, the tectonic history of the area results in a much more complicated geology than published literature indicates, particularly for the northern portion of the alignment.

From oldest to youngest, the bedrock formations that underlie the site include the Sequatchie Formation (Ordovician age), the Rockwood Formation (Silurian age), the Chattanooga Shale (Devonian age), the Fort Payne Chert and the Warsaw Limestone (both Mississippian age). Of those five formations, the Sequatchie Formation (Churnet, 1997) and the Warsaw Limestone have the least impact on the project, as they occur in very limited areas at the extreme southern and northern ends, respectively, of the alignment and only minor amounts of earthwork are proposed in those areas. The Chattanooga Shale is only shown to outcrop in a very limited band on published maps, but the borings indicate that it occurs outside of mapped areas, and its presence is a complicating factor as discussed later. The Rockwood Formation underlies about 25 percent of the alignment, but geologic considerations appear relatively straightforward with regard to this formation. The Fort Payne Formation is mapped as underlying the majority of the site, and has the more complicated geologic considerations. Fort Payne is shown as underlying the project alignment from the east flank of Stringers Ridge in the southern half of the project to almost the northern terminus of the alignment, about 60% of the project length.

Two separate thrust faults are mapped at the northern terminus of the project; those two faults merge to form a single thrust fault along the western toe of Laurel Ridge (on the north) and Stringers Ridge (on the south). That thrust fault marks the "slide plane" where the entire bedrock mass that makes up the subject ridge was shoved westward by tectonic forces long ago. The thrust fault is thought to angle ten to thirty degrees downward to the east, so it is present at relatively shallow depths beneath the entire project site. Because of the fault's proximity to the northern end of the project, its trace is quite shallow and the massive bedrock disturbance produced right along the fault is quite evident. Further away from the actual fault trace, those tectonic forces also resulted in folding and fracturing of the bedrock mass, such that bedrock dips, varying from fifteen degrees southeast to forty degrees northwest, are shown on published literature, and the bedrock cores encountered various dips ranging from horizontal to vertical.

DESIGN SEGMENTS

The information developed allowed the proposed alignment to be divided into five design segments based on similar subsurface conditions (Figure 3).

Segment 1

Segment 1 starts at the southern edge of the project at Sta. 2+536 and extends north to Sta. 3+100 (a length of 564 m). The general soil profile revealed at the boring locations consists of a veneer of topsoil and vegetation or paving overlying an interval of existing fill. The fill interval generally has a firm to very stiff consistency or, within the sand intervals, a medium dense to dense consistency. The fill within the right-of-way was placed to support the existing highway during its construction in the late 1950s or early 1960s. Beneath the fill is an interval of soft to stiff, silty clay with occasional traces of organic material that appears to be alluvial in origin, a result of deposition from the nearby Tennessee River. The alluvium extends to the refusal depths at the south end of the project. Groundwater was detected between three to five meters above the surface level of the Tennessee River is at approximate elevation 194 meters.

Nineteen of the 67 borings within this segment were drilled to refusal or encountered refusal shallower than their planned termination depths. Refusal depths ranged from 2.2 meters to 26.7 meters and averaged 15.1 meters below the existing ground surface. The remaining 48 borings were terminated at depths ranging from 3.1 meters to 14.4 meters and averaged 5.4 meters below the existing ground surface. Within Segment 1, two borings were cored to evaluate the refusal material. Bedrock, consisting of weathered, shaly limestone, was encountered at depths of 12.1 meters and 23.0 meters, respectively, below the existing ground surface at those cored locations. Neither of the cored holes lost drilling fluid during coring.

Segment 2

Segment 2 starts at Sta. 3+100 and extends north to Sta. 3+430 (a length of 330 m). The existing alignment within Segment 2 is predominantly a side hill cut/fill. Along the west side of the alignment, the highway is benched into the cut through Stringers Ridge. The east side of the alignment consists of fill embankments up to 13 meters high.

Within the eastern portion of the right-of-way, fill is present. The fill is composed of slightly sandy, silty clay with variable amounts of chert fragments. The fill is estimated to have firm to stiff consistency. A relatively thin interval of stiff to hard, residual, silty clay and clayey silt with shale fragments underlies the topsoil on the west side of the right-of-way, or underlies the fill on the eastern side of the right-of-way. Beneath the residual soil, augers were able to be advanced through a significant interval of hard silt or decomposed shale until auger refusal was encountered. Rock coring revealed that bedrock within this segment consists predominantly of weathered shale with occasional lenses and layers of shaly siltstone and hematite (Rockwood Formation). No ground water was detected during drilling or after 24 hours at boring locations within Segment 2.

Eight of the 18 boring within this segment were drilled to refusal. Refusal depths ranged from 11.3 meters to 33.0 meters and averaged 17.6 meters below the existing ground surface. The remaining 10 borings were terminated without refusing at depths ranging from 4.6 meters to 39.7 meters and averaging 11.6 meters below the existing ground surface. Within Segment 2, seven borings were cored to evaluate the refusal material. The core recovery (ratio of length of core recovered to total length cored) was moderate, (average of 72 percent), and the rock quality designation (RQD); the ratio of core lengths over 100 mm to total length cored was 13.5 percent (i.e., very poor) on average. Drilling fluid was lost in two of the core holes during drilling.

Segment 3

Segment 3 is a short reach about 140 meters long. Segment 3 appears to contain a fill at the head of a hollow, with an estimated maximum depth of 18 meters based on a review of topographic information. However, none of the borings extend the full depth of the fill, so we are uncertain of the actual fill thickness. The fill is composed of stiff to very stiff, sandy, silty clay that contains numerous shale and chert fragments. Two borings encountered a surface interval of topsoil underlain by stiff to hard, silty clay residuum. The silty clay gradually increases in consistency with increasing depth, and can generally be described as decomposed shale. The rock core for one boring revealed that the refusal material is composed of variably weathered shale. Water was measured in that same boring at a depth of 6.4 meters after at least 24 hours.

Two of the four borings within this segment were drilled to refusal. Refusal depths ranged from 14.9 meters to 16.3 meters and averaged 17.6 meters below the existing ground surface. The remaining two borings were terminated at depths of 4.6 and 18.5 meters below the existing ground surface without refusing. Within Segment 3, one boring was cored to evaluate the refusal material. The boring, located near the northern end of Segment 3, revealed that the bedrock is composed of weathered, black Chattanooga Shale that is present at a depth of 16.3 meters below the existing ground surface.

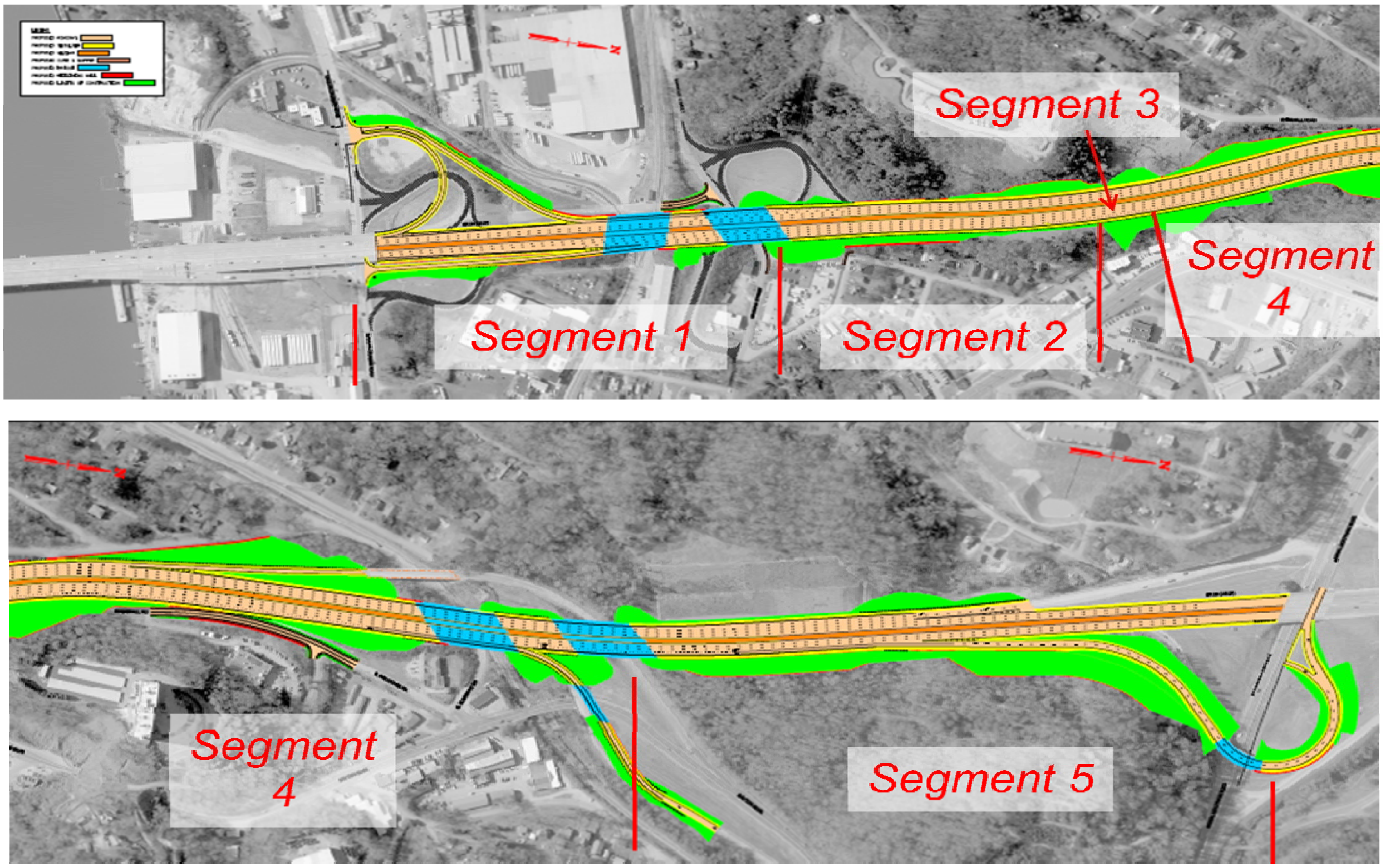

Figure 3 – Layout of Design Segments

Segment 4

The existing alignment within Segment 4 is predominantly a cut, up to 21.4 meters in depth, through Stringers Ridge. Beneath the surficial topsoil or pavement section, there are some isolated, relatively shallow, intervals of fill. The deepest intervals of fill are adjacent to the abutments at one of the bridges (Bridge 3). The fill is composed of stiff to very stiff, silty, sandy clay with varying amounts of chert fragments. Residual, silty, clay with varying amounts of chert fragments underlies the fill or surface interval of topsoil or pavement throughout Segment 4. The clay within Segment 4 is predominantly high plasticity clay (Liquid Limit > 50). In a few cases, the chert fragments are so abundant that the soil was visually classified as clayey, coarse sand with chert fragments. The consistency of the residuum ranged from soft to very stiff. The soil's consistency appears to vary depending upon the quantity of chert fragments in the soil as well as the moisture content of the clay matrix. Abundant chert fragments within the clay matrix tend to exaggerate the standard penetration test N-values in some samples. Furthermore, the laboratory tests show that some of the samples within Segment 4 have moisture contents above its plastic limit (PL), which means that the soil will likely deform if loaded. For instance, 21 of the 26 measured natural moisture contents within this segment are above the same soil's plastic limit, eight of those have moisture contents 10 points or more above the plastic limit.

Water was detected in four borings at the time of drilling in Segment 4. After monitoring for at least 24-hours, water levels were measures in a total of ten boring. Four of those borings, located on the east side of the alignment and one boring, located on the west side, appear to be retained drilling fluid. Additionally, water levels were measured in one boring at elevation 246.8 meters and in three borings near Bridges 3 and 4 at an average elevation of 213.0 meters after at least 24-

hours. Water was detected in another boring after monitoring for at least 24-hours, but was most likely surface water inflow from rainfall runoff.

Fifty-seven of the 87 borings within this segment were drilled to refusal. Refusal depths ranged from 4.2 meters to 28.5 meters and averaged 11.7 meters below the existing ground surface. The remaining 29 borings were terminated at depths ranging from 4.6 meters to 21.1 meters and averaging 5.2 meters below the existing ground surface without refusing. Within Segment 4, rock-coring techniques were used to advance 37 selected borings into refusal material. The rock core revealed that, in general, the bedrock is highly weathered to decomposed, siliceous limestone (Fort Payne Formation). The average core recovery was moderate (average of 55.1 percent), and the average RQD of 21.7 percent was very poor. Two borings had average RQDs above 75 percent. An additional four borings had average RQDs above 50 percent. Two borings had RQDs above 25 percent, and the remaining 29 borings had RQDs less than 25 percent. Seventeen of the 37 cored holes lost drilling fluid during drilling.

Segment 5

Segment 5 contains a very complicated subsurface regime and is predominantly located within existing cuts through Laurel Ridge, a set of northeast – southwest trending hills. The northern edge of this segment corresponds to the approximate limit of a secondary fault that roughly follows the southern edge of Signal Mountain Road (SR 8). The residual soils within this segment reflect the convoluted, twisted, and folded nature of their parent bedrock formations. Residuum within this segment includes cherty, silty clay derived from the Fort Payne Chert Formation; sandy, silty clay residuum of the Rockwood Formation; and black silt residuum from the Chattanooga Shale. The residual, silty, cherty, high plasticity clay interval derived from the Fort Payne Formation predominates throughout Segment 5. As in Segment 4, the chert fragments are occasionally so abundant that the soil was visually classified as clayey, coarse chert sand. The consistency of that residuum ranges from soft to very stiff. The soil's consistency appears to vary depending upon the quantity of chert fragments in the soil as well as the moisture content of the clay matrix. The occasional abundant chert fragments within the clay matrix tended to exaggerate the standard penetration test N-values in some samples. As in Segment 4, the laboratory tests show that some soils within Segment 5 have relatively natural moisture contents higher than the plastic limit, which implies plastic deformation is likely. Ten of the 12 measured natural moisture contents within this segment are above the same soil's plastic limit; four of those have moisture contents 10 points or more above the plastic limit.

The residuum of the Rockwood formation appears in portions of 18 of the 63 borings in Segment 5. The Rockwood residuum was found in borings located predominantly on the west side between Station 4+500 and 4+700 (10 borings). But, it was also found between Station 4+625 and 4+880 (6 borings) on the east side. Rockwood residuum in Segment 5 consists of soft to stiff silty clay or clayey silt with shale fragments. The intervals of Rockwood residuum appears to be situated at random elevations, and are, generally, not continuous or in proper order with respect to other identified residual soil units.

Residual soil identified as derived from Chattanooga Shale was detected with portions of 11 borings in Segment 5. The Chattanooga Shale residuum is typically composed of black, firm to stiff, very silty clay or clayey silt with variable quantities of decomposed to weathered shale fragments. That soil appears to be predominantly "ground up" shale.

Fourteen borings in Segment 5 encountered water during drilling, but water remained in only five of those borings after at least 24-hours. At those five borings, ground water elevations were between 199.7 and 210.1 m and were judged to indicate the general groundwater level in that vicinity.

Eighteen of the 63 borings within this segment were drilled to refusal. Refusal depths ranged from 0.9 meters to 20.7 meters and averaged 19.6 meters below the existing ground surface. The remaining 45 borings were terminated at depths ranging from 3.1 meters to 18.5 meters and averaging 11.5 meters below the existing ground surface. Within Segment 5, the rock core revealed that the bedrock is a chaotic mixture of rock types. The predominant rock types encountered by the borings include highly weathered to decomposed, siliceous limestone, weathered to decomposed pyritic, black shale, and weathered, light brown shale and siltstone. The average recovery within this segment was good (average of 81 percent) and the average RQD of 47.7 percent for the segment is considered poor. None of the cored holes lost drilling fluid during drilling.

GEOLOGIC HAZARDS

Karst

Much of the project is underlain by limestone formations, including the Fort Payne Chert, the Sequatchie Formation and, to a lesser extent, the Warsaw Formation. These bedrock types include limestone that it is highly soluble. Consequently, the limestone weathers deeply, usually producing a thick, cohesive soil interval overlying a highly irregular bedrock surface. Further, it is not unusual to find voids within the bedrock system and at the soil-bedrock interface. Many of the borings encountered soft, wet, residual soil at or above the bedrock surface and, also, soil filled and open voids to great depths within the bedrock. Additionally, although no closed depressions (sinkholes) were shown on the USGS topographic quadrangle there was a report of a prior karst related collapse in the immediate vicinity. Therefore, it was judged that there is at least a minor risk of future karst related collapse/sinkhole development. Surface water drainage would need to be carefully controlled to mitigate the risk.

Seismic Conditions

South central Tennessee has a record of numerous, small to very small earthquakes and the historical data suggests that the proposed project will have a low risk of being subjected to seismic accelerations of 0.10 g or more. That is based on seismic acceleration maps developed with 90 percent probability of ground motion not being exceeded in 50 years. Generally, seismic criteria did not govern the wall designs.

Landslides and Colluvial Soils

Within Hamilton County, most landslides occur in colluvium of the Pennington Formation, Fort Payne Formation, Chickamauga Group and Knox Group. Published reports (Hershey, 1979) depict landslides in the Fort Payne residuum and colluvium in the immediate vicinity of the project (e.g., south end of the Stringers Ridge Tunnel entrance). As noted previously, there are several landslides within the project site. To evaluate the activity of the largest apparent slide, inclinometer casings were installed in borings are located in an apparent slide between Stations 3+720 and 3+850 on the east side of the right-of-way. After initialization, the casings were monitored for about four months. Other than some slight movement that apparently occurred while the casing was seating, no significant trend was observed during the monitoring period.

Colluvial soil was not identified in any of the borings. However, identifying colluvium is, in most cases, difficult because it resembles, to a great extent, the parent soil from whence it came. It is likely that there are some colluvial deposits within the project limits.

Additionally, although most rock units are below the depth of excavation, the regional dip of the bedrock formations appears to result in the bedding planes of the geologic formations sloping downward toward the proposed roadway cuts along the eastern side of the highway (i.e., the slopes facing west). Adverse dips may also occur at any location along either side slope along Segment 5 due to the severely contorted geologic beds.

DESIGN CONSIDERATIONS

To facilitate the proposed construction, Bridge 3 was widened and Bridges 1, 2, 4 and 5 were replaced. Fill walls of up to 10 m in height were required to support bridge approaches, ramps, and prevent filling of adjacent and existing roads. Much of the rest of the alignment required either extensive sloping or edge-of-road retaining walls in fill areas as well as cut walls up to 15 m in height to support the slopes.

Fill walls were designated by the letters A through M. Cut wall locations were designated with a combination of letters and numbers (i.e., W1, W2, W3, and S1 through S5, etc.). Supplemental retaining walls were required behind most of the cut walls to achieve adequate slope stability safety factor for the slopes above the primary walls. This "tiered" system of walls was required to reduce required excavation, support weak residual soil slopes and limit right-of-way acquisition.

Cut Wall Design Parameters

During the exploration, recovery of a testable, relatively undisturbed sample was successful only about one in every ten attempts. The poor recovery was, in large part, due to the high chert content of the soils. It became necessary to employ alternative methodologies in order to confirm soil strength parameters.

Of the 16 triaxial tests performed, five multistage triaxial tests (Sridharan and Rao, 1972) were conducted to estimate the shear strength parameters of a single sample of a soil. The 11 consolidated-undrained (CU) with pore pressure measurement triaxial tests had a mean effective angle of internal friction (φ') of 21.5° with a coefficient of variation (COV) of 0.32; mean cohesion was 10.4 kPa with a COV of 1.56. The coefficient of variation is defined as the standard deviation divided by the mean (Baecher and Christian, 2003). The mean φ' of the five multistage triaxial tests was also 21.5° with a COV of 0.22; mean cohesion was 20.0 kPa with a COV of 1.22.

If an existing slope demonstrates signs of instability or existing failure conditions, limit equilibrium back analyses provide a fairly accurate φ' estimate that is representative of the whole slope. In addition to triaxial tests, back analyses were performed on 12 of the existing slopes assuming a factor of safety of unity within each of the design segments. The mean f˚ from back analysis was 23.7° with a COV of 0.14. Cohesion was assumed to be zero.

Published studies have also addressed correlation of soil classification and index tests, such as Atterberg limits and the associated Unified Soil Classification System designation, to soil strength parameters. The laboratory test results generated during this study and the back analyses that were performed, were compared to published correlations (*Figure* 4) in order to verify that the results are reasonable and prudent.

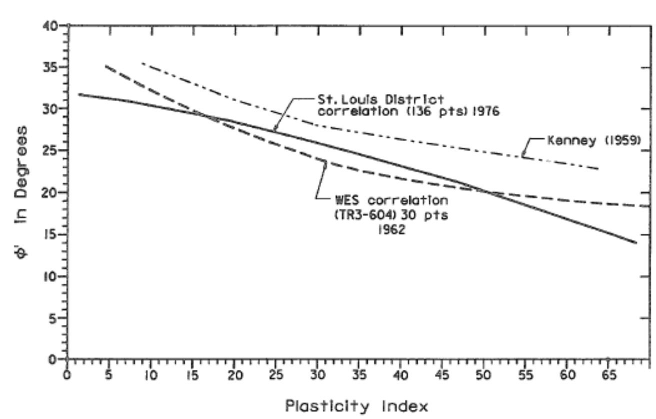

Figure 4 - Drained Friction Angle Versus Plasticity Index
(after Fig. 2-4, USACE, 1989)

The Kenney (1959, Eq. 1), St. Louis District (1976, Eq. 2), and WES (1962, Eq. 3) correlations were digitized and regressed into the following equations:

$$\varphi' = 0.0031 \times PI2 - 0.4433 \times PI + 38.939 \qquad \text{(Eq. 1)}$$

$$\varphi' = -0.0018 \times PI2 - 0.1465 \times PI + 32.029 \qquad \text{(Eq. 2)}$$

$$\varphi' = 0.0042 \times PI2 - 0.5565 \times PI + 37.306 \qquad \text{(Eq. 3)}$$

The correlation results were compared to the measured values and it was determined that the plasticity index correlations generally over-estimated the drained friction angles (Figure 5).

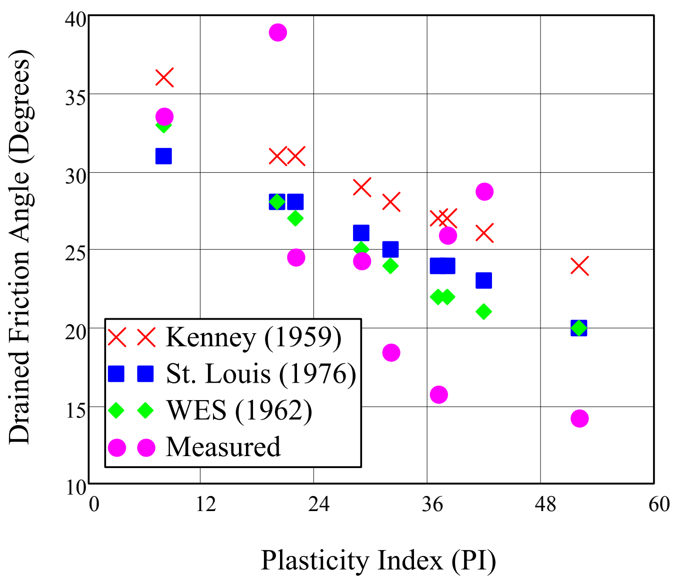

Figure 5

However, when the Kenney, St. Louis, and WES correlations were averaged together and compared to the measured results, the measured/computed drained friction angle ratio was 0.94 with a COV of 0.26.

Based on the approach taken, drained friction angles were assigned to each of the in-situ soils at the fill and cut walls for design. The values assigned, ranged from 21˚to 30˚ with an average value of 25˚ and a COV of 0.11.

<u>Cut Wall Apparent Pressure Diagrams</u>

The proposed cut walls consisted of drilled-in H-piles with tieback anchors, temporary lagging, and a cast-in-place concrete facing in an ashlar pattern (*Figure* 6). The facing was attached to the wall by welded Nelson studs embedded in the concrete.

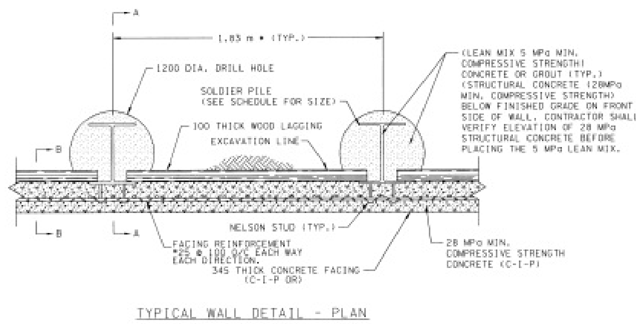

Figure 6 – As Designed

Because of the cast-in-place facing, the cut walls were designed as tall, stiff soldier pile walls using the combined pressure diagram shown in *Figure* 7. Using this backwall pressure in analysis presented a wall system with limited required maintenance, which was preferred by the Owner.

Note: Fill walls were designed by the Contractor and will not be discussed further in this paper.

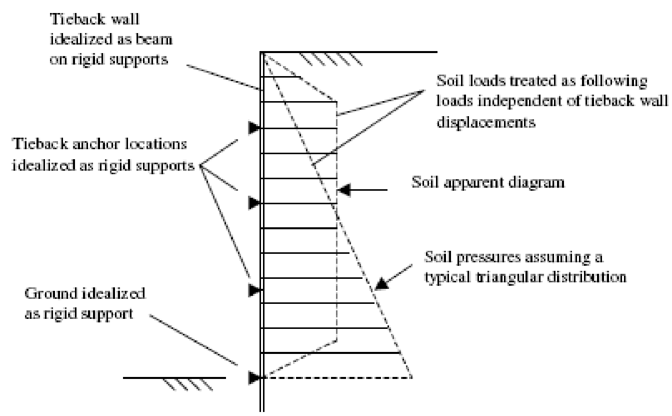

Figure 7 - Apparent Pressure Diagram
(Strom and Ebeling, 2001, Figure 1.3 RIGID Method)

The apparent pressure diagram was used as a starting point in the design process to determine preliminary anchor location and loads. Cross-sections were drawn and a limit equilibrium slope stability analysis was performed to check global stability assuming the assigned in-situ soil strengths and an assumed, allowable bond strength between the ground and the anchor grout of 85 kPa.

A target factor of safety for long-term, drained global stability of the cut walls was set at 1.5 and the anchor number and lengths were modified to achieve the target value. The most significant constraint for the anchor lengths was that the anchors had to be installed within TDOT right-of-way.

Once the design sections were analyzed with two-dimensional limit equilibrium methods, a staged-construction finite element analysis (FEA) was performed using PLAXIS. This was done to study the effect of excavation, anchor installation and stressing, and long-term consolidation/creep on the walls. Determination of the required embedment depth of the soldier piles was determined using both an LPILE-type analysis along with the PLAXIS analysis.

CONSTRUCTION MONITORING

The FEA indicated that movement would occur during excavation and anchor stressing. Inclinometers were installed behind each cut wall to monitor wall deflection. Project specifications required that continuous monitoring be performed by the contractor for a minimum of six weeks after excavation and anchor stressing were complete to ensure that facing was not cast until after detectable movements had terminated. This process was put into place to mitigate cracking of the cast-in-place facing.

ENGINEERING DURING CONSTRUCTION

Prior to the bid-letting in February of 2012, the soldier pile configuration was altered from a traditional “double-pile” element. *Figure* 6 shows an example of the soldier pile layout as designed; *Figure* 8 shows the fabricated soldier pile. The design consisted of a single H-pile with each anchor location requiring a hollow structural steel (HSS) tube welded to the web and flanges. The loss of section modulus was mitigated by the use of cover plates on the front and back of each pile.

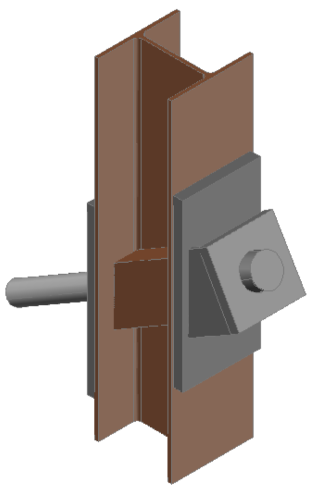

Figure 8 - Fabricated Soldier Pile

The HSS and the cover and wedge plates were designed by selecting trial sizes of each and performing a FEA using Bentley’s RAM ADVANCE software (*Figure* 9). The size of the structural elements were adjusted until the working stresses of the steel materials were within allowable limits.

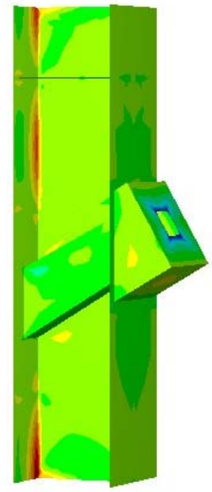

Figure 9 – FEA of Fabricated Soldier Pile

Soldier pile drill holes were sized according to the LPILE analysis and required to be 1200mm diameter. This allowed the fabricated pile is able to be installed in the soldier pile drill holes complete. The contractor proposed to use face-mounted wood lagging, using the shear studs as the means of connection to the pile. The shear studs are the only elements required to be field welded to the pile.

CONCLUSIONS

The soldier pile wall system selected was adaptable to the multiple geologic settings along the project. However, selecting an adaptable wall system is not enough. Due to the variable nature of the project geology, a comprehensive monitoring program is essential for determining the appropriateness of the engineered system. The monitoring program reinforces the Owner’s preference for a limited maintenance wall system. Construction and monitoring to date have performed within or better than design requirements.

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