Soil Retention System Analysis & Redesign

Executive Summary of Analysis

The Dulles Pedestrian Walkback Tunnel project used a soil retention system composed of shotcrete and tiebacks to maintain the sides of the excavation during construction. While this is a temporary system and will not be used once foundation walls are completed, the design and function of this wall is crucial to the success of the project.

Problems were encountered early in the excavation phase of the project, which resulted in many delays and change orders due to differing site conditions and severe weather conditions. A recurring problem on site was the collapses of the excavation before the retention system could be constructed.

An alternate system was selected, a slurry wall. The slurry wall was designed to withstand the passive soil pressure loading throughout construction.

Construction durations and cost estimates were calculated, and was found to reduce the overall duration of earthwork activities as well as the overall cost.
Overview

All projects need to provide adequate support and protection to the adjacent structures. In this project, the excavation is located very close to an existing building, Concourse B, and therefore the support and protection of the existing facilities is of the utmost importance. During the course of the excavation, not only will the support system, caissons, of Concourse B be exposed, but they will need to be braced to remain structurally sound.

Surrounding the rest of the site, East, North and West sides, a minimal area will be allocated for site work. No setback is allowed for this project, therefore the selection of a soil retention system is limited. The options for the soil retention system on this project are:
- Braced walls using wales and struts
- Soldier beam and lagging
- Braced sheeting
- Bored-pile walls
- Diaphragm-slurry walls

The drawings and specifications did not specify a particular soil retention system to be used. The civil engineer’s original design incorporated piles and lagging and also shotcrete with tiebacks. Upon consultation with Berkel & Company Contractors (BCC), Hensel Phelps Construction Company (HPCC) elected to use a soil retention system made up solely of shotcrete and tiebacks, abandoning the use of piles and lagging. This system was chosen based on the rocky sub-grade conditions and desire for excavation to be waterproofed before foundations began.

Design Constraints

Evaluating these systems to find the ideal system for the project is fairly simple due to the site constraints. The constraints are as follows:
- No setback allowed
- Below grade conditions consist of weathered, fractured rock with veins of organic and clay
- Heavy surface surcharge loads (600 PSF during construction)
- Existing Concourse B adjacent to the South
  - 3 caissons will need to be braced and supported
- Groundwater was at a higher elevation than originally anticipated
- All groundwater removed from the site must be carbon filtered prior to disposal
- Due to organic layers in the rock, small cave-ins are frequent if unsupported
- Excessive vibration is unacceptable due to the proximity of existing structures

Post-Construction Analysis

The selection of the shotcrete and tiebacks system was based on certain assumptions from the geotechnical report. These assumptions included the overall quality of the rock below grade, and the groundwater conditions. The rock was assumed to be able to support itself within 6-8’ lifts. However, once excavation reached 10’ below grade on the West side of the site, minor collapses occurred, halting all work in the area. In order to continue in this zone, piles were driven in order to stiffen the soil, improving the overall quality. This was a very expensive measure that could not be performed throughout the rest of the project. As excavation continued, the rock remained very fractured with veins of organic soils only perpetuating more cave-ins.
Another issue arose due to the amount of water needing to be removed from the current excavation. Extreme amounts of rain in the summer of 2003 required nonstop dewatering for weeks. Since all water removed from site is contaminated, it must be filtered through a slow and costly process.

As the project progressed, problems were encountered on the East wall at about 25’ below grade. Very deep and wide collapses due to a combination of the fractures in the rock and the layers of the organic-clay mixture caused more stoppages in the process. Additional designs needed to be performed due to the additional depth (nearly 3’ in some cases) in order to correct the shotcrete wall system. With extra shotcrete, reinforcing, and a few weeks, the process was again underway. While the excavation was originally scheduled to begin in February and end in
May, it did not actually begin until March and was not finished until the end of October, an eight month duration.

Since these problems were differing site conditions, and fall under the DSC clause, change orders were issued amounting to nearly $700,000 due to flooding and earth movement. Change orders in this process doubled the original cost of the soil retention system to reach roughly $1.5 million.

**Selection of an Alternative System**

In an attempt to find a better system, it is necessary to evaluate the various soil retention options. To do this, a matrix was formulated, the system on one axis, and the key issues on the other axis.

<table>
<thead>
<tr>
<th>Issues</th>
<th>Sloped/Stepped Sides</th>
<th>Braced Walls using Wals and Struts</th>
<th>Soldier Beams and Lagging</th>
<th>Braced Sheeting</th>
<th>Sheetin w/ Tiebacks</th>
<th>Bored-Pile Walls</th>
<th>Diaphragm-Slurry Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Setback can not be accommodated on site</td>
<td>NOT POSSIBLE</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
</tr>
<tr>
<td>Weathered, fractured rock with veins of organic and clay</td>
<td>Use Rock Grinder</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
<td>Interferes w/ Rock Grinder</td>
<td>Use Rock Grinder</td>
<td>NOT POSSIBLE</td>
<td>Use Hydro-Mill</td>
</tr>
<tr>
<td>Collapses imminent if unsupported</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
</tr>
<tr>
<td>Heavy surface surcharge loading</td>
<td>NOT POSSIBLE</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
</tr>
<tr>
<td>Existing building on South side of site</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
<td>OK</td>
<td>OK</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
</tr>
<tr>
<td>Groundwater - high water table</td>
<td>NOT POSSIBLE</td>
<td>PUMP</td>
<td>PUMP</td>
<td>PUMP</td>
<td>PUMP</td>
<td>PUMP</td>
<td>MIN. PUMP</td>
</tr>
<tr>
<td>Vibrations to existing structures</td>
<td>OK</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
<td>NOT POSSIBLE</td>
<td>OK</td>
<td>OK</td>
</tr>
</tbody>
</table>

From this analysis, it can be seen that a sloped or stepped excavation is not possible for a number of reasons, mostly due to the constricted site surrounded by active airline gates, and the below grade conditions. Braced walls using wales and struts are also out of the question due to the need for support of the sides of the excavation throughout the excavation process.

A system composed of sheeting and tiebacks has only one key constraint, however vibrations pose a serious problem due to the existing structures adjacent to the site. The slurry wall has one constraint, which poses a significant problem. If a slurry wall system was used, an alternate system would need to be used on the south side of the site in order to work beneath the overhang of the existing Concourse B.

Evaluating the use of a slurry wall on the rest of the site, it appears to be a valid alternative. A setback is not required for the construction and use of a slurry wall. The weathered, fractured rock can easily be ground by a hydromill, an attachment used for deep trenches that is lowered into the trench with a mobile crane. Collapses will be avoided through the use of the slurry. A heavy surcharge load can be accounted for in the design of the thickness.
of the wall, as well as the tiebacks. Finally, the slurry wall will be advantageous when encountering groundwater seeping into the site, as a concrete wall one foot thick will be fairly impervious to water.

Design of an Alternative System

The design of a slurry wall is approached in a similar manner to the design of a braced sheetpile retention system. Tiebacks are used in the same manner, with a substitution of a concrete wall instead of a sheet of steel.

The first step in design was to determine the width of the wall, or the stem width. A width of 15” was used as a preliminary dimension. However the minimum width of a slurry wall is dependent on the method of excavation of the trench. While the wall could in fact be designed as 15” to withstand the forces, it would never be able to be built. Thus the width was increased to 36” to allow a hydromill to be lowered by a mobile crane. The hydromill will serve to grind up the rock within the trench. The spoils within the trench will be removed by an excavator with a 60’ stick, the same excavator used on site for the shotcrete system.

The width analyzed to ensure it was greater than the minimum width requirement for the wall. To reduce the risk of exposed reinforcement, the slurry wall will be designed to ensure 3” of cover for all reinforcing. An increased cover is necessary due to the wall being cast against earth on both sides. A width of 10” from the outer edge of reinforcing to the edge of the wall was calculated to ensure proper the wall could withstand the applied shear forces. A spacing of 3” on the side, plus 10” in the middle, adds up to a 13” minimum thickness for the wall. A 36” width wall will be sufficient for this application.

STAAD, a structural analysis and design program, was used to calculate shear forces and overturning moments for the wall. Instead of modeling the entire wall in STAAD, a unit width of 1’ was modeled to simplify the model and use the program more efficiently. Modeling a 1’ unit width strongly resembles a beam, only oriented vertically, not horizontally.

In order to account for the passive soil pressures on the wall, a distributed load was applied to the beam. The distributed load was calculated according to the Terzhagi and Peck Trapezoidal Apparent Earth Pressure Diagram (P=0.29γH), as shown. This method was recommended by the geotechnical engineer in the soils report.
The nodes of the “beam” were defined according to the type of restraint on the beam. Tiebacks will be used as the primary restraining force for the slurry wall. Tiebacks will be used at the same locations as were designed for the original shotcrete system, which consists of a 7’ horizontal spacing, and a 6’-7’ vertical spacing.

The tiebacks will act as pin connections in the system; no load will be placed upon the tiebacks vertically, or across the wall, only through the wall. The only other type of connection to be modeled in the system is the resistance provided by the earth at the bottom of the excavation. The slurry wall is constructed so as the base of the wall extends further than the base of the excavation. This gives the wall additional structural support, as the “connection” at the base of the excavation now acts as a fully fixed connection, a moment connection.

During the course of excavation, the wall will be exposed from the top, down. Therefore it is important to analyze the impacts of the varying soil pressures on the wall. To account for this, a STAAD model was generated for 7 different scenarios. For example, the first scenario
encountered would result in a 6’ depth, with node #1 at grade level, and node #2 at the current bottom of excavation, as shown.

Node #1 is not restrained in any direction, since no tieback will be installed at grade level. Node #2 has been restrained as a moment connection.

The next scenario will be at a depth of 12’, with a node at grade level, a node at 6’ where a tieback has been installed, and a node at the current bottom of excavation. The moment connection has moved from node #2, to node #3 as the bottom of the excavation as moved deeper. Node #2 has been changed to a pin connection due to the tieback restraining the wall, as shown.
These scenarios continue until the final bottom of excavation is reached. Each scenario was analyzed, and it was determined that the last scenario would present the greatest loads on the wall. The following image and chart displays the complete depth of the system.

**WORST CASE SCENARIO**

**STAAD RESULTS**

<table>
<thead>
<tr>
<th>“BEAM”</th>
<th>NODE</th>
<th>SHEAR (kip)</th>
<th>MOMENT (ft-kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>3.915</td>
<td>-7.830</td>
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<tr>
<td>3</td>
<td>3</td>
<td>5.160</td>
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<td>4</td>
<td>4</td>
<td>5.566</td>
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<tr>
<td>5</td>
<td>5</td>
<td>5.918</td>
<td>-6.716</td>
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</tr>
<tr>
<td>7</td>
<td>7</td>
<td>6.946</td>
<td>-8.443</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>5.146</td>
<td>6.152</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.309</td>
<td>-2.228</td>
</tr>
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</table>

The reinforcing designed for the wall with the results from the STAAD analysis; calculations can be referenced in the Appendix. This design process is very similar to the design of a retaining wall’s stem. The shear strength of the wall is calculated using the values of maximum shear to ensure the wall is designed to the proper thickness.

The required cross sectional area of the vertical reinforcing steel is calculated using the values of the maximum moments attained from the modified STAAD results. Reinforcing steel must meet a minimum ratio, as defined by code, to ensure excessive cracking does not occur. Once the vertical reinforcing steel is designed, the horizontal steel can be checked. Code requires a minimum cross sectional area of horizontal reinforcing.
The calculations result in a wall design as shown.

**DESIGN OF REINFORCING**

**PORTION OF WALL ELEVATION**

**SECTION VIEW**

**REINFORCING**

**REINFORCING**

Vertical reinforcing for the wall consists of two rows of #6 bars at 8” on center. Two different scenarios were analyzed for the design of the horizontal reinforcing. This is necessary due to the distributed load applied to the wall in between the tieback elevations. The load will be transferred vertically to the elevation of the nearest tieback, then transferred horizontally through the reinforcing to the nearest tieback. Minimum horizontal reinforcing was also calculated, and was found to control the design of the horizontal reinforcing. This results in two rows of #6 bars at 12” on center throughout the height of the wall. Additional reinforcing was included within the design to ensure sufficient strength of the wall if a bar is cut in the process of drilling a tieback.
Progressing through the design process, the next step is the design of the tiebacks. Although tiebacks will not be designed in this analysis, much of the information needed is already available from calculations thus far.

Since a unit length of this beam was taken as 1’, all calculations output from STAAD must be multiplied by 7 for the design of the tiebacks, to account for the actual tributary area of the soil pressure on the tiebacks, as seen shown.
The modified results, the STAAD results multiplied by a factor of seven, are as shown.

<table>
<thead>
<tr>
<th>&quot;BEAM&quot;</th>
<th>NODE</th>
<th>SHEAR (kip)</th>
<th>MOMENT (ft-kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>27.405</td>
<td>-54.810</td>
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<tr>
<td>2</td>
<td>3</td>
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<td>3</td>
<td>3</td>
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</tr>
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<td>4</td>
<td>4</td>
<td>38.962</td>
<td>39.634</td>
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<td>5</td>
<td>5</td>
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<td>-47.012</td>
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<td>5</td>
<td>6</td>
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</tr>
<tr>
<td>6</td>
<td>6</td>
<td>49.182</td>
<td>59.101</td>
</tr>
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<td>7</td>
<td>44.604</td>
<td>-43.064</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>36.022</td>
<td>43.064</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>2.163</td>
<td>-15.596</td>
</tr>
</tbody>
</table>

The next step in the process is the design of the tiebacks. The design requirements can be calculated from the end forces on each beam. However since the tiebacks are spaced horizontally 7’ on center, the STAAD output for shear must be multiplied by 7’ as shown above. The angle of the tiebacks will be assumed to be drilled at the same angle as the original shotcrete system. The conditions for bond length for the tie shall also be assumed the same.

**Schedule Impacts**

The construction of a slurry wall in place of the shotcrete wall will have major impacts on the sequencing and scheduling of the project. The slurry wall must be one of the first activities started on the project, due to a need for the slurry wall to be fully completed before the excavation activities can begin. The construction of the shotcrete wall had four concurrent activities; rock grinding, removal of spoils, spraying shotcrete, and installing tiebacks. A 6’ lift would take the entire crew approximately one week to complete, barring any stoppages due to collapses, excessive groundwater, or other problems on site.

The construction of the slurry wall is estimated to take approximately three weeks. This duration was reached through consultations of several industry professionals at Shaw Environmental, Dorfman Construction, and Bauer Equipment. This three-week duration consists of one week for mobilization, one week for excavation of the trench, and one week for demobilization.

Mobilization for slurry wall construction is fairly intensive, including the delivering of a mobile crane, a hydromill, an excavator, and equipment for the mixing of bentonite slurry. This equipment will use most of the area on the site, therefore attempting to start other activities will only result in increased crowding and possibilities for accidents.
The excavation of the trench for the slurry wall can be estimated at approximately 100 linear feet per day at a depth of 50’. Approximately 400 linear feet of wall will be needed on site, resulting in 4 days of construction. An extra day is added to account for possible delays.

During the construction of the slurry wall, another contractor will be on site to place rebar cages into the trench, and pump the concrete to form the panel. Since alternating panels will be constructed, crowding will not be a concern for these two subcontractors.

Demobilization will take approximately one week. The loading and removal of the mobile crane and hydromill, as well as the removal of the bentonite slurry and storage containers will make up the major aspects of this process.

Once the slurry wall is complete, excavation can begin. While the excavation for the project will be delayed an additional three weeks while the slurry wall is constructed, the increased efficiency of the excavation process will make up for the initial delay. The excavation process will be similar to the original process, however there will be no need for the constant repositioning of equipment in order to spray shotcrete on the walls. The tiebacks will still need to be installed, however this can be completed by two or three men on site, as compared to six or seven for the complete shotcrete and tiebacks process.

By eliminating the shotcrete operation, the duration for a 6’ lift decreases by two days. Fewer operations on site allow for two rock grinders to be used, which makes full use of the excavator and loader. A lift cycle time of three days over the course of eight lifts results in a total duration of five weeks for excavation operations. Adding the initial three weeks for the construction of the slurry wall, results in an overall duration of eight weeks, or two months.

In summary, using a slurry wall retaining system as compared to the shotcrete retaining system saves six months.

Cost Impacts

Through consultations with several industry experts at Shaw Environmental, Dorfman Construction, and Bauer Equipment, it was concluded that a unit cost of approximately $8-10 per square foot would be a reasonable assumption of cost of the slurry wall for the situation at the Connector B site. 400 linear feet of wall, 50’ deep results in 20,000 square feet of slurry wall, resulting in a total cost of approximately $200,000. This price does not include the concrete and rebar for the wall itself, therefore additional estimates are necessary. Approximately 2000 CY of concrete and 1500 tons of rebar will be used within the slurry wall system, adding up to a cost of approximately $250,000.

The rock grinding on site is unchanged from the original process, and is valued at approximately $400,000. The removal of all spoils from the excavation is valued at approximately $500,000. This accumulates to a total cost of $1,350,000 for the excavation and soil retention of Connector B.

This estimate must be taken in context, these are not bids therefore the costs may be stated too low. However, when comparing the cost of a slurry wall system to the shotcrete retention system with a total cost of nearly $2,000,000, significant cost savings appear to be possible.
Other Impacts

Safety
Through the use of a slurry wall in place of the wall of shotcrete, many potential hazards are eliminated. The previous concern of minor collapses is no longer an issue, as the slurry wall will ensure the perimeter does not collapse.

Fewer workers will be on site during the excavation phase, which reduces the chances of an accident or injury occurring. The shotcrete system required 10-12 workers to be on site, 6-8 for the shotcrete application, 2 for the grinding operation, and 2 for the removal of spoils.

The slurry wall system would use the same number of workers, but since the slurry wall would be complete before the excavation could begin, a maximum of 6 workers would be on site at any one time. During the construction of the slurry wall, 4-6 workers would be required on site. Once complete, the excavation process would begin consisting of 2 workers for the grinding operation and 2 workers for the removal of spoils.

Quality
The surface of the slurry wall would certainly be rougher than the surface of the shotcrete wall. The tiebacks for the shotcrete wall could be covered and embedded within the wall itself. This is important for the application of the waterproofing which cannot be applied to any sharp surfaces.

The surface of the slurry wall is likely to be more jagged due to the fractured nature of surrounding rock. Tiebacks will also need to be on the surface of the wall, due to their installation after the slurry wall is complete. The waterproofing details should be analyzed and modified to determine an alternate detail at these places of difficulty.

Foundation Walls
The foundations for Connector B consist of 4'-3” thick foundation walls on all sides of the excavation. The original process on site for the construction of these walls utilized one sided forms, and the shotcrete/waterproofed wall as the other side of the form. During the procurement process of the one sided forms, it was realized that the design would need to be stronger and more intricate than originally anticipated, driving up the cost of the forms. The added complexity of the design was needed due to the large hydrostatic pressure exerted on the forms of 4'-3” thick concrete extending upwards 26 feet. The possible solutions resulted in the pinning of the formwork to the matte slab below, as well as bracing the formwork and pinning the braces to the matte slab. The added cost of materials and labor was unanticipated on site, and would be a cost burdened by the general contractor.

A possible solution arises for this situation through the design of the soil retention system. If the slurry wall can be designed in a way to allow a form-tie to be used, significant cost savings could be seen through the decreased complexity of the formwork. Since the concrete wall itself cannot provide the needed tensile strength to hold the tie in place, another component must be used. Tiebacks are already being used in the system, therefore some efficiency would be seen if the same component can be used for the form-tie connection.

Though the additional tiebacks add labor and time to the soil retention system, significant savings will be seen on the erection of the formwork system. Far less bracing and pinning will be required, and the formwork assembly will be standardized for each lift of the foundation
walls. As a result, productivity on site will be increased, decreasing the overall duration of setting formwork, and increasing the profitability of the operation.

A cost analysis of the extra tiebacks to be installed in order to use a cheaper formwork system would be the next step if this is option is to be pursued.

**Future Construction**

Construction for another tunnel adjacent to the West side of the connector is planned in the near future. This tunnel will house a train to give passengers a third option of transportation to and from the Main Terminal and Concourse B.

The construction of Connector B is planned in a manner to anticipate this future expansion of the tunnel level of the building. Foundation walls have been designed to allow for simple local demolition of necessary walls. However the shotcrete retention system has not been designed in a manner to assist with this process.

The construction of a slurry wall inherently produces vertical panels around the perimeter of the site. If the needs for the future expansion are analyzed early enough in the design process, these particular panels could be design to assist with the demolition efforts, without changing the design of the rest of the retention system. This allows for a more efficient design, and cost savings for the next package of Walkback Tunnel.

**Conclusions**

A shotcrete retaining system is best used in situations of constricted sites of moderate quality rock, as the grinding of rock will result in a flat surface for the application of shotcrete. As an alternative, a slurry wall retaining system can be used in situations of poorer quality rock or soil, but quite constricted site conditions.

Careful considerations must be made in order to design and implement a slurry wall retaining system. The design differs greatly from the design of a shotcrete system, as does the sequencing and scheduling of the excavation phase on site. While the schedule may appear to lengthen due to necessity of completing the slurry wall prior to the removal of spoils, once excavation is underway, the process will be much faster compared to the shotcrete system.

In this situation on the Pedestrian Walkback Tunnel, the slurry wall retaining system is recommended for the site of Connector B.