Fugro Consultants, Inc.
2880 Virgo Lane Tel: 972-484-8301
Dallas, Texas 75229 Fax: 972-620-7328

North Tarrant Express (NTE) *3A*

Fugro Technical Memo No. 3

To: Jiri Filipovic, PE – AECOM
Anthony Karlinski, PE - AECOM
Carlos Gonzales, PE- Ferrovial Agroman (FA)
Oscar Aguirre, PE – Aguirre & Fields, Inc.

From: Saad M. Hineidi, P.E. & Chula B. Ellepola, P.E.
Date: December 21, 2009
Fugro Project No.: 04-4009-1033

Re: Preliminary Geotechnical Recommendations for Drilled Shaft, Soldier Pile, & Soil Nail Retaining Walls – **Cut Sections**

This design memorandum presents geotechnical information for the design of drilled shaft, soil nail, and tied-back HP soldier pile retaining walls proposed for the NTE Segment 3A project. These types of retaining walls are proposed for those cut sections where it is not possible to construct mechanically stabilized earth retaining walls (MSE RW) due to the lack of lateral area to construct the required MSE select backfill volume or where it is necessary to keep the traffic lanes open. Our recommendations are based on the following:

- Plan and profile drawings provided by AECOM,
- Cross section drawings provided by AECOM,
- Critical sections identified and communicated to Fugro by Aguirre and Fields, Inc.(A&F), and
- Subsurface information obtained through the advancement of 22 bridge and pavement borings by Fugro Consultants, Inc. (Fugro).

This information is preliminary and is aimed at providing the design team with adequate information to dimension the drilled shaft, soldier pile, and soil nail retaining walls and tiebacks to develop a bill of quantity (BOQ). Additional borings and analyses are required to develop final geotechnical design recommendations.

Discussion

We recommend using drilled shaft or soil nail retaining walls to support cut roadway sections and to facilitate top-down construction. We understand that a retention system comprising of tied-back drilled soldier pile walls with concrete lagging is another option being considered. MSE retaining walls are better suited for bottom-up construction. According to the Federal Highway Administration (FHWA) Engineering Circular No. 7 and personal conversations with contractors and designers, soil nail retaining walls are the least expensive and preferable system. We understand that a soldier pile/beam and a precast lagging retention system is the second 2nd cheapest system. Although a soil nail retaining wall with a height of about 30 feet has been successfully constructed, we recommend limiting the height of soil nail retaining walls to about 20 feet or less. Soil nail retaining walls could be used in the following conditions:

- In low and moderately plasticity clays;
- In weathered rock;
- In soils and rock where there is an adequate standup time (does not slough);
- When a lateral distance of at least 100 percent of the wall height is available behind the soil nail retaining wall;
- Where the soil nail retaining walls are constructed above the groundwater level.

Soil nail retaining walls should not be used in the following situations:

- In loose fills and sandy soils – will not have adequate standup time;
- Below the groundwater level;
- When construction takes place during temperatures of 40 degrees Fahrenheit and falling;
- Where there will be utility line and easement conflicts;
- In expansive clays due to potential creep problems; and
- Where there will be large surcharge loads.

Drilled shaft and/or soldier pile and lagging retaining walls could be used in those areas where it is not possible to use soil nail retaining walls. There are numerous, experienced drilled shaft contractors in the Dallas-Fort Worth metropolitan area and they may be able to construct drilled shaft retaining walls quicker and more economically than a soil nail retaining wall. Although, soldier pile retaining walls are less commonly used in this area, the use of this type of cut wall is becoming more popular. The respective specialty contractors should be consulted to determine the relative costs, completion schedules, and construction limitations.

The subsurface profiles for this project should be superimposed on the cross sections provided by Earth Tech and A&F to perform the preliminary analyses needed to develop the BOQ.

Drilled Shaft Retaining Walls

We provided geotechnical recommendations for the axial design of drilled shafts in the Fugro Technical Memorandum No. 1 dated December 7, 2009. A&G told us that cut walls are being considered in the general vicinity of Station Nos. 778+00 and 786+00. Obviously, the preferred wall system is a soil nail wall system, followed by a soldier beam and tie-back wall system, followed by a drilled shaft retaining wall system.

According to the information provided to us, cut retaining wall systems are expected to retain cut heights up to 21 to 23 feet in the general vicinity of Sta. 778+00 and 786+00. The drilled shaft retaining wall will structurally act as a cantilevered beam restraining the soil and rock materials above the base of the excavation and transferring the restrained loads to the bearing strata below the base of the excavation. The bearing strata are expected to be gray limestone, shale, or a combination of both. Consequently, the drilled shafts will be subjected to the active lateral pressures of the restrained soils if the drilled shafts experience lateral movements on the order of 0.01 to 0.002 times the height of the wall. If the retaining walls are restrained, they will be subjected to the at-rest lateral earth pressures, which are greater than the active lateral pressures. Therefore, the magnitude of the lateral pressures will be a function of the stiffness of the drilled shafts and the type of soil being restrained. For preliminary design purposes, we are recommending the use of a weighted average of 60 percent active lateral pressure and 40 percent at-rest lateral pressure for the most commonly encountered materials at the project site. These pressures will act as a triangular pressure and are presented in Table 1. These values are based on the results of the triaxial tests, directs shear tests, unconfined test results, and TCP test results.

Table 1. Lateral Triangular Pressures For Drilled Shaft Walls – Weighted Average

Soil Type	Moist Unit Weight, pcf	Coefficient of Lateral Pressure	Lateral Pressure Without Hydrostatic Loading	Lateral Pressure With Hydrostatic Loading
			Equivalent Fluid Pressure, pcf	Equivalent Fluid Pressure, pcf
Fat Clay Fill, Moderately Plastic Fill, Native Moderately Plastic Clay, Native Fat Clay	125			
-Horizontal Backfill		0.57	71	98
-3H:1V Slope		0.61	76	101
Low Plasticity Sandy Clay/Clayey Sand	125			
-Horizontal Backfill		0.51	64.5	94
-3H:1V Slope		0.56	69.5	97
Sand				
-Horizontal Backfill	125	0.46	58	91
-3H:1V Slope		0.50	63	94
Shale				
-Horizontal Backfill	130	0.30	39	83
3H:1V Slope		0.35	46	86
Gray Limestone				
-Horizontal Backfill	140	0.20	28	78
3H:1V Slope		0.24	34	81

All surcharge loads should be multiplied by the respective lateral earth pressure coefficient and added to the equivalent fluid pressure to determine the total lateral earth pressures acting on the retaining walls. Typically, TxDOT uses a traffic surcharge pressure of 250 psf. For aesthetic purposes, pre-cast panels are installed on the exposed face of the drilled shaft retaining walls. To facilitate drainage and to prevent the occurrence of hydrostatic pressure, synthetic vertical drains are typically installed between the drilled shaft walls and the pre-cast panels.

Lateral load analyses will be needed to compute the bending moments, the shear forces, deflection, and shaft head deflections of the drilled shafts. Obviously, the bending moments, shear forces, and deflections of the drilled shafts are functions of the wall height, concrete compressive strength, section modulus (EI), and steel reinforcement. Typically, the taller the wall, the greater are the moments, forces, and deflections of drilled shaft.

If the lateral loads cause large, unacceptable bending moments, shear forces, and deflections in the drilled shaft, the bending moments, shear forces, and deflections could be reduced by using *tie-back anchors*. The use of tiebacks will cause a redistribution of the lateral pressures and will subject the drilled shafts to a "*trapezoidal*" pressure. For tall retaining walls, the "*trapezoidal*" pressure will result in greater lateral forces and moments on the drilled shafts. For preliminary design purposes, the "*trapezoidal*" pressure diagram is presented on Plate 1 of this report. The preliminary geotechnical design recommendations for the tieback anchors are presented later in this report.

Resistance Against Lateral Loads For Drilled Shaft Retaining Walls

Because the drilled shafts will have a relatively large slenderness ratio (length divided by diameter), it is likely the drilled shafts will be subjected to flexural bending because of the lateral loads. To evaluate the lateral resistance, the drilled shafts can be modeled as a beam on an elastic foundation using the Winkler spring concept to establish the stress-strain relationship of the materials. These parameters are substituted into the Hetenyi fourth order differential equation establishing the relationship between the moment, stress, displacement, and soil properties. Computer programs (such as L-Pile, PY Wall, or COM624) solve this equation using the finite difference method and tabulate the moments, displacements, and shear forces with depth in the output. For this type of an analysis, we recommend using the following parameters in the analysis.

Table 2. Geotechnical Design Parameters For Laterally-Loaded Drilled Shafts

Material	Clays	Sandy Clays	Sands	Shale	Gray Limestone
Horizontal Subgrade Modulus, K kips/in^3	1.0	0.5	0.04	2.0	5.0

Table 2. Geotechnical Design Parameters For Laterally-Loaded Drilled Shafts

Material	Clays	Sandy Clays	Sands	Shale	Gray Limestone
ε_{50}, in/in	0.005	0.004	-	0.003	0.001
Cohesion, ksf	0.6	0.2	0	2.5	5.0
Angle of Internal friction, °	0	12	26	30	40
Loading Option	Static	Static	Static	Static	Static
Moist Density, pcf	125	125	125	130	140

Alternatively, for preliminary design purposes, the lateral resistance of the drilled shaft can be computed by static equilibrium using the passive pressures of the soils, shale, limestone, or both shale and limestone to resist the lateral pressures. We recommend using a rectangular-distributed allowable passive resistance of 300 psf in the clays, clayey sands, and sands; 2,000 psf for the shale; and 10,000 psf for the gray limestone. The lateral resistance values are valid for soils below a depth of 5 feet below final grade, for shale below a minimum penetration of 3 feet, and for gray limestone below a minimum penetration of 2 feet. These values are for closely-spaced drilled shafts that have a clear distance varying from 3 to 12 inches between each other.

Vertical ground settlements approaching the magnitude of the maximum drilled shaft lateral deflection should be expected to occur behind the drilled shaft retaining wall. These settlements should occur within a lateral distance equal to about 75 to 100 percent of the wall height from the back of the wall. The impact of these deflections on underground utility lines, pavements, etc. within the influenced zone should be carefully considered. Additional engineered fill equal to the magnitude of the anticipated settlement could be placed within the impacted zone to compensate for the estimated settlement.

Preliminary Geotechnical Design of Tiebacks for the Soldier Pile and Lagging System

The following figure shows the profile of the probable layout of the proposed tiebacks:

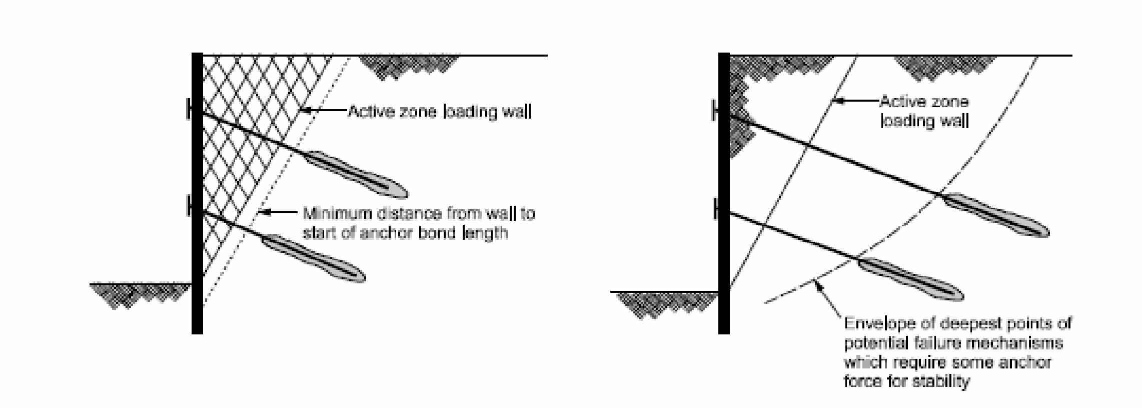

Figure 1. ***Cross sectional view of the probable tiebacks.***

As shown in Figure 1, the tieback will consist of an unbonded section and a bonded section. The tieback force will be derived from the frictional interaction of the grouted/bonded section with the surrounding subsurface materials. It is quite possible that the bonded section of the tieback could pass through the fat clays, sands, shale, and the limestone. Tiebacks are generally installed at an angle of 15 to 30 degrees below the horizontal and extend to a lateral distance of about 2 to 3 feet short of the right of way (ROW). The design parameters for the tiebacks extending through these materials are based on FHWA methodology (Table No. 7 FHWA Circular No. 4) in conjunction with our test results. These parameters are presented in the following table.

Table 3. Tieback Design Parameters

Material	Allowable Bond Stress, psf
Fat Clay	600
Sandy Clay	750
Sand	1,200
Shale	6,000
Gray limestone	10,000
Note: A safety factor of 2 was used to compute the allowable bond stress.	

As shown on Figure 2, there should be a minimum linear distance of 0.2 times the height of the wall between the edge of the potential failure surface and the grouted/bonded section. For preliminary design, assume that the failure surface starts at the base of the base and is inclined upwards until it intersects the ground surface at an angle of 45 degrees from the vertical.

The bond force is computed as product of the length of the bonded section, the average circumference of the grouted annulus, and the allowable bond stress presented in Table 3. As shown in Figure 1, a global stability analysis is generally performed for the tiebacks and the grouted sections are placed behind the failure surface. Because specific information about the configuration of the retaining wall, the subsurface soils and rock, the diameter of the proposed tie-backs, etc are required for global stability analyses, these analyses will be performed during the final geotechnical investigation when the pertinent information is available.

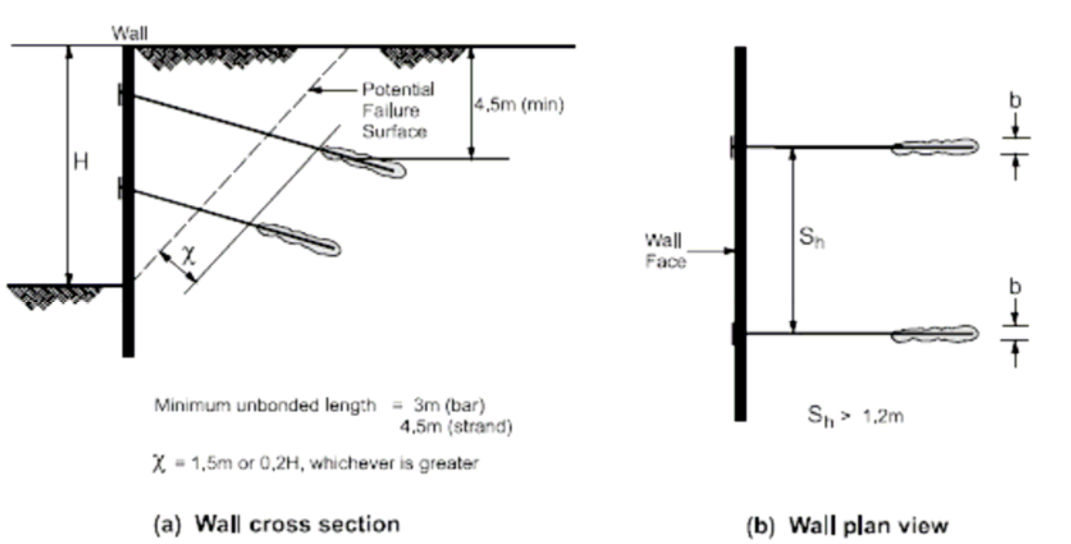

Figure 2. ***Minimum Distance Between the Failure Surface and the Start of the Grouted Section.***

Figure 2 provides some guidelines about the placement and sequencing of the tiebacks. The dimensions are in meters. Meters could be converted to feet by multiplying a meter by 3.28 feet. The considerations associated with tiebacks used for resistance typically include:

1. general stability of the enclosed groundmass;
2. changes in the anchor load , which can occur as a result of movements within the groundmass, such as excavation heave, consolidation, swell, and creep; and
3. corrosion protection.

We strongly recommend performing several pull-out to failure tests to accurately evaluate the tieback capacity since the tieback capacity is highly depended on the subsurface materials and the tieback contractor's workmanship. We recommend that the specialty contractor be pre-qualified based on his previous experience for similar conditions. The contractor should be required to submit an installation plan with a proposed schedule, sequence, and list of equipment to be used for the project.

A construction testing program that includes proof, performance, and creep testing of representative anchor installations would be required to help confirm that the anchors will perform as intended. In general, each anchor should be proof tested. Performance and creep tests should be performed on 10 percent of the anchors. The contractor should submit a test grout mix for approval prior to the start of construction. The grout cubes should be obtained on a daily basis to help confirm that the compressive strength of the grout is satisfactory.

A comprehensive construction monitoring and record-keeping program is recommended to help reduce risks associated with improper construction methods. The construction monitoring should be performed by qualified personnel independent of the contractor and should include:

- Anchor hole diameter and length, drilling equipment and methods;
- Soil and rock types, ground conditions;
- Anchor hole cleanout, cave-in;
- Tendon dimensions, condition of protective layers, centralizers;
- Anchor grouting, pressure, volumes, post-grouting;
- Anchorage installation, alignment, strand elongation, lock-off;
- Waterproofing, trumpet grouting;
- Record of above observations;
- The test results and construction reports should be monitored and evaluated on a daily basis during installation, and
- Other related items.

Another method of anchoring the drilled shaft retaining wall to reduce the bending moments, shear, and deflection of the drilled shafts is to install a second row of drilled shafts behind the original drilled shaft retaining walls and structurally connect the two rows of drilled shafts with structural beams. This method would provide greater resistance than a deadman anchor.

Preliminary Geotechnical Design of Soil Nails

A soil nail wall is constructed by drilling an inclined hole, installing a steel bar, and grouting the hole. As shown in Figure 3, the soil nail wall consists of steel reinforcing bars, grout, nail heads, hex nuts, washers, bearing plates, temporary and/or permanent shotcrete facing, welded wire mesh (WWM), geocomposite strip drainage, and corrosion protection.

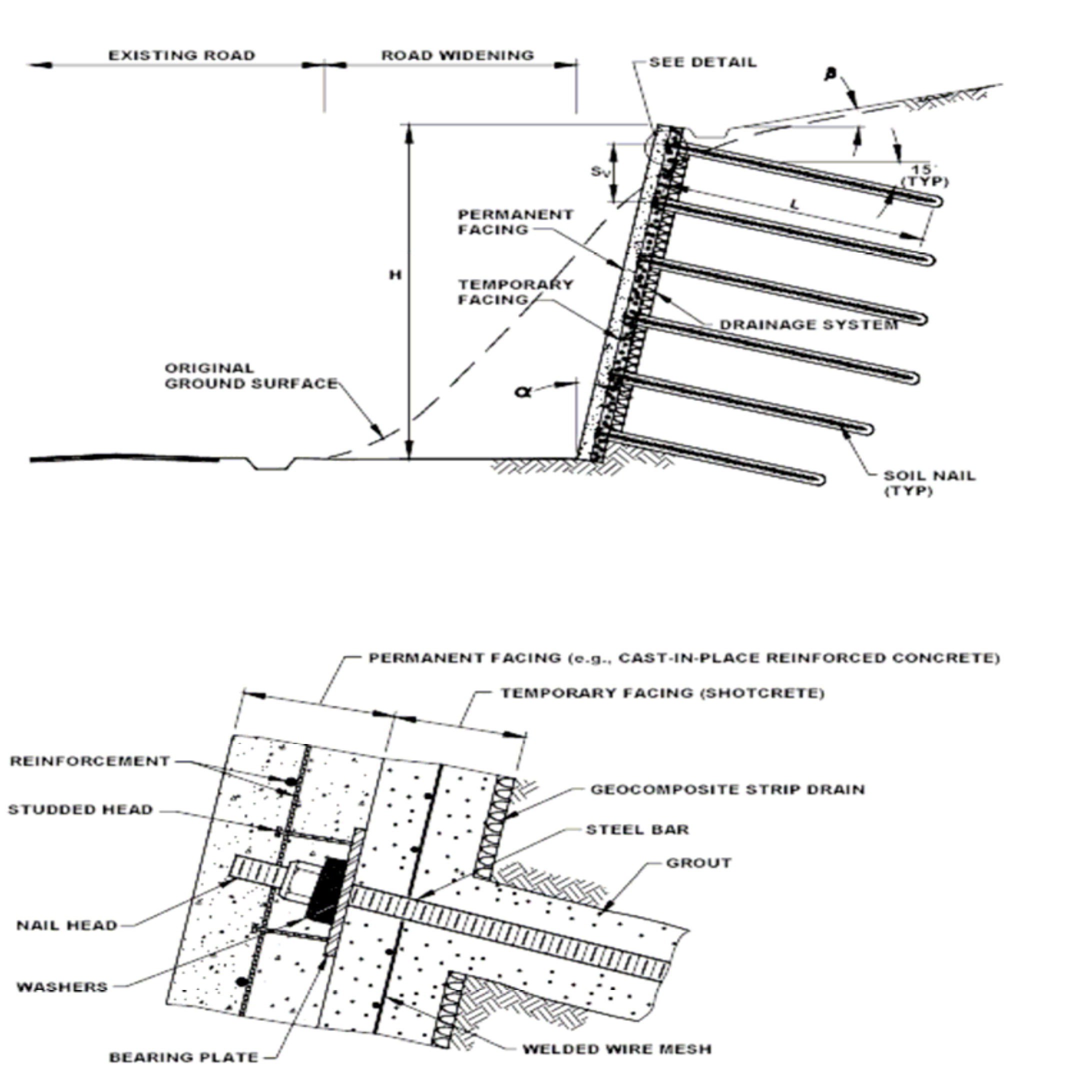

Figure 3. ***Typical cross section of a soil-nail wall.***

Because soil nail walls are passive, they must experience some lateral movement to generate the resistance to further lateral movement. Thus, the soil nail walls will be subject to the active lateral earth pressures, which are triangular. These values are presented in the following table.

Table 4. Lateral Active Pressures For Soil Nail Walls

Soil Type	Moist Unit Weight, pcf	Coefficient of Lateral Pressure	Lateral Pressure Without Hydrostatic Loading	Lateral Pressure With Hydrostatic Loading
			Equivalent Fluid Pressure, pcf	Equivalent Fluid Pressure, pcf
Fat Clay Fill or Native Fat Clay				
-Horizontal Backfill	125	0.51	64	94
-3H:1V Slope		0.55	69	97
Sandy Clay/Clayey Sand				
-Horizontal Backfill	125	0.45	56	91
-3H:1V Slope		0.49	61	93
Sand				
-Horizontal Backfill	125	0.39	48	87
-3H:1V Slope		0.43	54	89
Shale				
-Horizontal Backfill	130	0.27	35	81
3H:1V Slope		0.30	39	83
Gray Limestone				
-Horizontal Backfill	135	0.18	25	76
3H:1V Slope		0.21	29	79

All surcharge loads should be multiplied by the respective lateral earth pressure coefficient and added to the equivalent fluid pressure to determine the total lateral earth pressures acting on the retaining walls. Typically, TxDOT uses a traffic surcharge pressure of 250 psf. For aesthetic purposes, pre-cast panels are installed on the permanent or temporary shotcreted face of the soil nail retaining walls. To facilitate drainage and to prevent the occurrence of hydrostatic pressure, synthetic vertical drains are typically installed between the retained soil and the shotcrete facing.

As mentioned earlier, the soil nail system is a passive system that needs some lateral soil movement (0.1 to 0.2 times the wall height) to develop resistance. Thus, the resistance of soil nails is computed as the length of the bonded section extending beyond the failure plane multiplied by the average diameter of the grouted annulus multiplied by the allowable bond stress. The allowable bond stress for soil nails extending through these materials are presented in Table 5. These values are based on the values recommended in Table 3.10 of FHWA Geotechnical Engineering Circular No. 7 (Soil nail design and construction) in conjunction with our interpretation of the results of our in-situ and laboratory strength test results.

Table 5. Soil Nail Design Parameters

Material	Allowable Bond Stress, psf
Fat Clay	600
Sandy Clay	1,000
Sand	1,000
Shale	4,100
Gray Limestone	5,000
Note: A safety factor of 2 was used to compute the allowable bond stress.	

The soil nail wall system should be designed to be safe for the external failure mode, the internal failure mode, and the facing failure mode. The external failure mode consists of checking the soil nail system for global failure, sliding failure, and basal heaving failure. We understand that the structural engineer will perform these analyses. We are providing the design parameters to perform the global stability and sliding failure analyses in Table 6.

Table 6. Global Stability and Sliding Parameters for Soil Nail Design

Material	Moist Weight, pcf	Effective Stress Parameters		Total Stress Parameters	
		C', psf	Φ', °	C, psf	Φ, °
Fat Clay	**125**	0	21	800	0
Sandy Clay	**125**	20	24	150	19
Sand	**120**	0	30	0	30

Table 6. Global Stability and Sliding Parameters for Soil Nail Design

Material	Moist Weight, pcf	Effective Stress Parameters		Total Stress Parameters	
		C', psf	Φ', °	C, psf	Φ, °
Shale	**135**	1,000	18	2,000	0
Gray Limestone	**140**	5,000	45	5,000	45
Note: For the sliding coefficient, use tan (2xΦ'/3) of the material with the lowest Φ' value.					

The spacing of the soil nails in the horizontal and vertical directions is about 5 feet on center and a square pattern of placing the soil nails is generally used. The nail lengths are typically between 70 to 100 percent of the wall height. The shotcrete facing is typically about 4 inches thick. The threaded solid steel bars (nails) that are commonly used are Grade 60 or Grade 75 Nos. 8, 9, 10, 11, and/or 14 bars. The grout for the soil nail walls consists of neat cement grout conforming to Portland cement Type I or II. Normally, a compressive strength of 3,000 psi at 28 days is specified for the grout. The ideal width of the WWM mesh reinforcement panel for a center-to-center vertical and horizontal nail spacing of about 5 feet should be about 5.5 feet. The cross-sectional area and mesh opening of the WWM must satisfy the structural requirements of the flexural and punching shear capacities. The maximum soil nail wall deflection is generally at the mid height of the wall.

For preliminary design purposes, the ground settlements of the soils behind the soil nail retaining walls will be about 25 percent more than that of the drilled shaft retaining walls. The impact of these deflections on underground utility lines, pavements, etc. within the influenced zone should be carefully considered. Additional engineered fill equal to the magnitude of the anticipated settlement could be placed within the impacted zone to compensate for the estimated settlement.

Preliminary Geotechnical Design of Tied-Back Soldier Pile Retaining Walls With Lagging

We understand that consideration is being given to using soldier pile retaining walls. We understand that the soldier pile walls will be constructed by drilling 24-inch diameter holes at

6-foot centers along the retaining wall alignment into the bearing stratum. Then, a HP 12x53 section will be dropped into the hole and centered. The annular space around the centered HP section will be filled with flowable grout. The grouted section will be allowed to set. As the excavation progresses from the top down, the grout will be removed from the flanges and 6-foot wide concrete lagging will be constructed between the flanges of the adjacent HP sections using the shotcrete method. The concrete lagging is to be solid at the flanges, which is estimated to be about 12 inches in thickness. Between the adjacent HP flanges and over the general span, the thickness of the concrete lagging is estimated to be on the order of 8 inches. The concrete lagging will be reinforced with vertical and horizontal rebar.

For shallow cuts, the soldier pile and lagging retention system is anticipated to act as a cantilevered system. Therefore, the shallow soldier pile and lagging retention system will be subjected to the active, triangular soil lateral pressures presented in Table 4 of this report.

To limit the wall deflections and provide additional resistance for deeper cuts, the soldier pile and lagging retention system will have to be restrained with one to several rows of tie-back anchors. The deep soldier pile and lagging retention system restrained with tie-backs will be subjected to "*trapezoidal*" lateral soil pressures. For tall retaining walls, the "*trapezoidal*" pressure will result in greater lateral forces and moments compared to the cantilevered design. For preliminary design purposes, the "*trapezoidal*" pressure diagram is presented on Plate 1 of this report. The preliminary geotechnical design recommendations for the tieback anchors were presented earlier in this report.

Table 7. Geotech Design Recommendations for Soldier Pile and Lagging Retention Systems

Wall Height ft.	Description	Lateral Pressure	Resistance
0-6	Cantilevered HP 12x53 Soldier Pile with No Tie-backs	Lateral pressures shown on Table 4	Values shown on Table 2 or use the passive resistance values presented in the "*Resistance Against Lateral Loads For Drilled Shaft Retaining Walls*" section of this report
6-12	HP12x53 soldier piles with 1 tieback placed at a depth of 5 feet from the top of the HP12x53	Trapezoidal Pressure Shown on Plate 1	
12-24	HP12x53 soldier piles with 2 tiebacks installed as follows: 1st tieback 5 feet from the top of the HP section; 2nd tieback placed at a depth varying from 3 to 10 feet below the 1st tieback.		
24-30±	HP12x53 soldier piles with 3 tiebacks placed as follows: 1st tieback placed 5 feet from the top of the HP section; 2nd tieback placed between 7 and 10 feet below the 1st row, and the 3rd tieback installed at a depth varying from 7 to 10 feet below the 2nd tieback.		

The retaining wall longitudinal and cross sections should be superimposed on the project subsurface profiles to perform the preliminary analyses needed to develop the BOQ.

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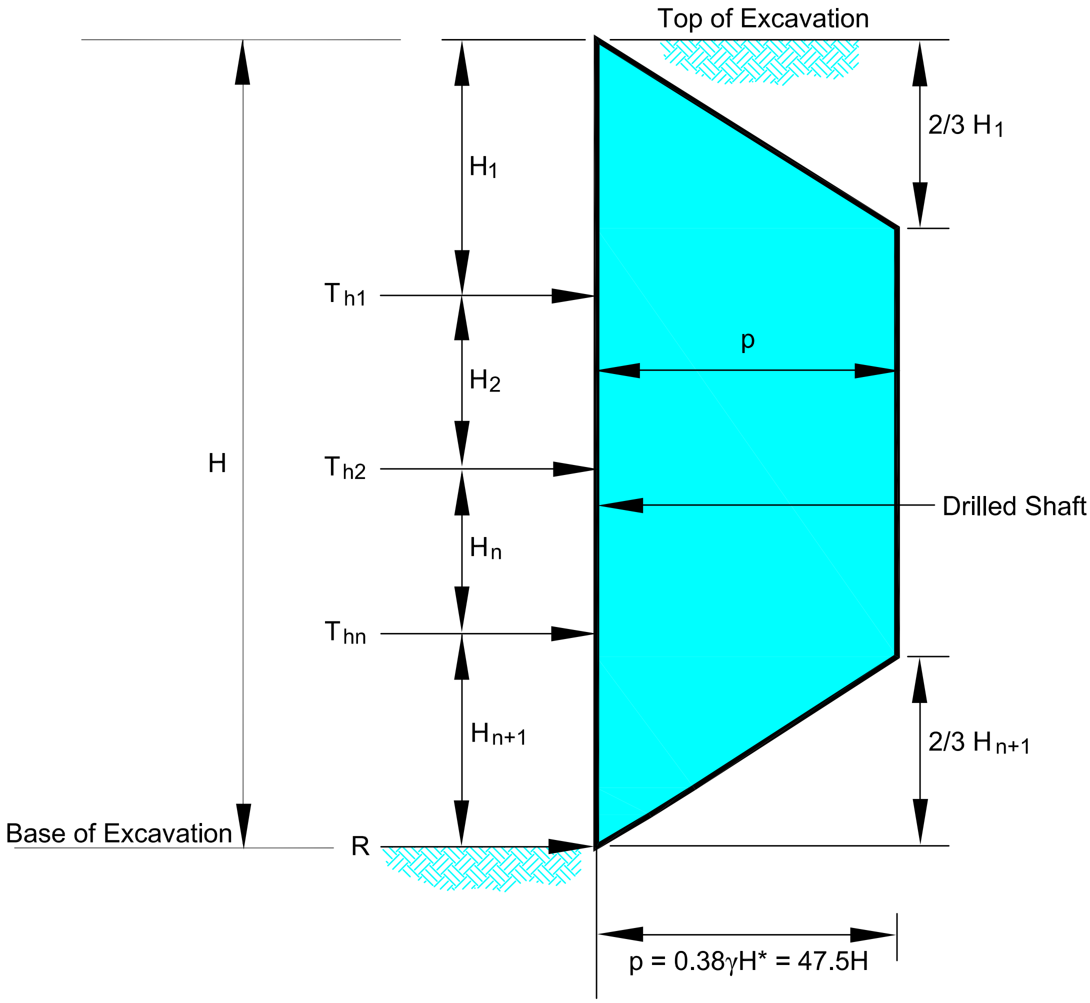
Additional design recommendations concerning all preliminary aspects of the design and construction of the soldier pile, drilled shaft, and soil nail retaining walls and the tie-backs will be provided in our geotechnical report to be produced later. In preparation of this memorandum, we have strived to perform our services in a manner consistent with that level of care and skill ordinarily exercised by other members of our profession currently practicing in the same locality under similar conditions. No other representation, express or implied is included or intended in this report, any addendum report, opinion, document, or other instrument of service.

ATTACHMENTS

The following plate is attached to this memorandum:

1. Plate 1. The “trapezoidal” pressure diagram for permanent tied-back drilled shaft retaining walls.



H_1 = Distance from ground surface to uppermost ground anchor

H_{n+1} = Distance from base of excavation to lowermost ground anchor

T_{hi} = Horizontal load in ground anchor i

R = Reaction force to be resisted by subgrade (i.e., below base of excavation)

p = Maximum ordinate of diagram

γ = 125 pcf

* Reference: FWHA Geotechnical Engineering Circular No. 4

NOT TO SCALE

Fugro Consultants, Inc.

RECOMMENDED EARTH PRESSURE ENVELOPE FOR PERMANENT TIED-BACK WALLS

NTE - Segment 3A | **Fort Worth, Texas**

Source:	Drawn By:	Date:	Project No.:	
Fugro Consultants, Inc.	DF	December 21, 2009	04-4009-1032	**PLATE 1**