December 21, 2023

STRUCTURE FOUNDATION EXPLORATION REPORT

MOT-725-14.41

RETAINING WALL RW-3

Montgomery County, Ohio

ODOT PID No. 108619
JMT Job No. 18-02295-001

Submitted to:
Ohio Department of Transportation, District 7

Prepared For:

Ohio Department of Transportation
District 7
1001 Saint Marys Avenue
Sidney, Ohio 45365

Prepared By:

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December 21, 2023

Mr. Dan Grilliot, P.E.
District Geotechnical Engineer
Ohio Department of Transportation, District 7
1001 St. Marys Avenue
Sidney, Ohio 45365

RE: **Structure Foundation Exploration Report**
MOT-725-14.41 Retaining Wall 3
Montgomery County, Ohio
JMT Job No. 18-02295-001

Mr. Grilliot,

Johnson, Mirmiran & Thompson, Inc. (JMT) is pleased to submit the results of the geotechnical subsurface exploration and geotechnical engineering recommendations for the above referenced project. This report contains a discussion of our understanding of the project, exploration procedures, the exploration results, and our geotechnical recommendations for the design and construction of Retaining Wall 3.

This report replaces all information and design pertaining to Wall 3 contained in the June 20, 2023 Structure Foundation Exploration Report 'MOT-725-14.41 Retaining Walls RW-1 Through RW-4'.

It has been a pleasure to be of service to you on this project. If you have any questions or need further information, please do not hesitate to contact us at this office.

Very truly yours,

JOHNSON, MIRMIRAN & THOMPSON, INC.

Steven Sommers

Steve Sommers, MS, PE
Geotechnical Engineer

EXECUTIVE SUMMARY

The Ohio Department of Transportation (ODOT) has proposed an interchange improvement project (MOT- 725-14.41, PID 108619) for State Route 725 (SR -725) and associated ramps with Interstate Route 75 (IR-75) in Montgomery County, Ohio. The overall project objective is to reduce the congestion and improve safety at the existing interchange at IR-75 and SR 725, as well as adding sidewalk alongside SR725 and upgrading the traffic signal at SR-725 and Byers Rd. The improvements proposed to accomplish this objective include: 1) the reconstruction of SR-725 between Byers Road and Mall Woods Drive; 2) the construction/reconstruction of the associated ramps; and 3) the construction of the retaining wall along the south side of SR-725 beneath the twin bridges as well as up to three other retaining walls along the ramps.

The subsurface profile within the proposed project area generally consists of surficial materials comprised of asphalt and base, generally underlain by natural stiff to hard cohesive soils and loose to dense granular soils. The natural stiff to hard cohesive soils encountered at the site of retaining walls consists of Sandy Silt (A-4a), Silt (A-4b) and Silt and Clay (A-6a). The loose to dense granular soils consists of Sandy Silt (A- 4a), Silt (A-4b), Course and Fine Sand (A-3a), Gravel and Stone Fragments with Sand (A-1-b), Stone Fragments with Sand and Silt (A-2-4) and Stone Fragments with Sand, Silt and Clay (A-2-6). Bedrock was only encountered in the historical borings near RW 1.

Wall 3 is 450.0 feet long and extends from SR 725 WB BL Sta. 461+45.18, 34.5' Rt. to Sta. 466+02.72, 25.11' Rt. (Wall Stas. 3500+00 to 3504+50.00). It is located approximately 40 feet to the right of the SR 725 CL. Wall 3 is to be a soil nail wall that starts to the west of the IR 75 SBL bridge over SR 725, traverses under IR 75, and finishes to the east of the NBL bridge over SR 725. The wall will create room for the pedestrian path running along the south side of SR 725. The maximum height of the exposed wall is approximately 9.2 feet occurring at Wall Sta. 3501+20. Soil profiles were developed to analyze/design the soil nail and concrete facing for the wall. Wall profiles, elevations, wall details, plan notes for the wall, shotcrete and wall facing concrete, and nail layout and loads have been included in the report.

Table of Contents

1.0 INTRODUCTION

This project originally consisted of the improvement of 0.44 miles of State Route 725 by widening to construct a diverging diamond interchange. Included in this project was the installation of pedestrian and bicycle facilities along State Route 725 and an upgrade on the traffic signal at Byers Road. Also included in this project were four retaining walls along SR 725: a cantilever concrete semi-gravity wall (RW-1), two soldier pile and lagging walls (RW-2 and RW-4), and a soil nail wall (RW-3). See Figure 1 for the locations of these walls. ODOT has decided to change the type of interchange which will greatly affect the overall project. However, the pedestrian path and RW-3 will remain unchanged. This report covers RW-3, while RW-1, RW-2, and RW-4 are presented in a separate report.

Johnson, Mirmiran & Thompson, Inc. (JMT) has performed the calculations and analyses to design RW-3. This report presents the results of these analyses.

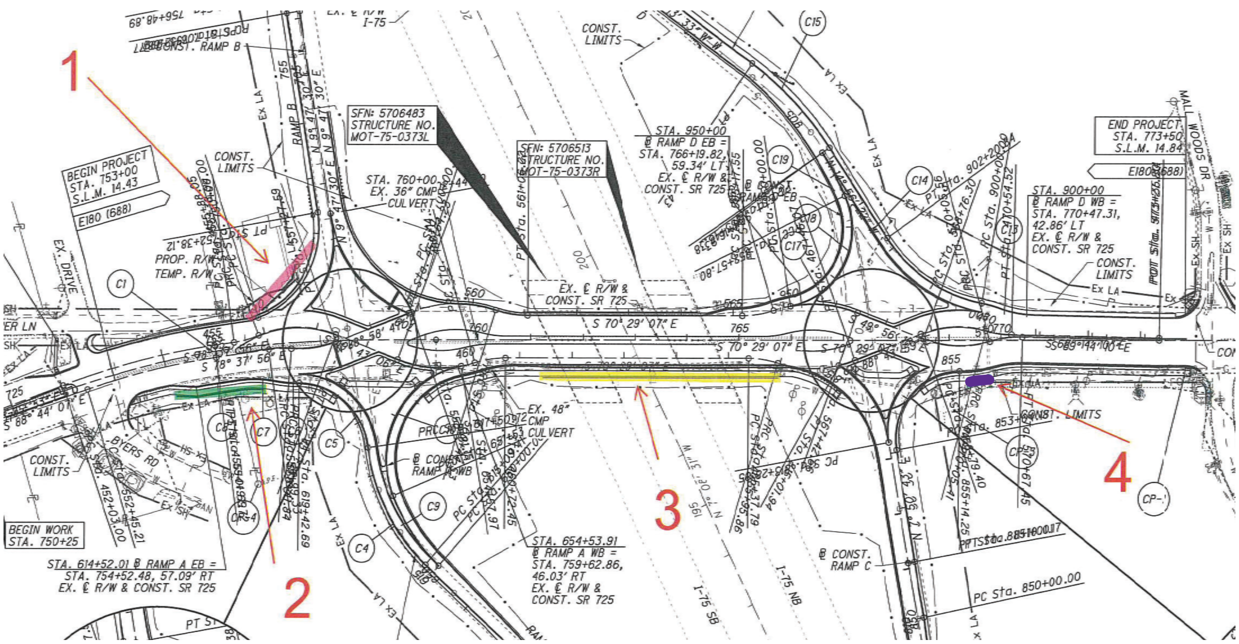

Fig. 1 Wall Locations

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

The project site is located within the Southern Ohio Loamy Till Plain which is characterized as end and recessional moraines, commonly associated with boulder belts, between relatively flat-lying ground moraine, cut by steep-valleyed large streams with surface soils consisting of loamy till. Buried valleys are common and are generally filled with outwash and alternate between broad floodplains and narrows. Elevations of the region ranges from 530 to 1,150 ft amsl, with moderate relief (200 ft). The geology within this region is described as loamy, high-lime Wisconsinan-age till, outwash and loess over Lower Paleozoic- age carbonate rocks (i.e., limestone or dolostone) and, in the east, shales. (ODGS, 1998).

Based on the Quaternary Geology Map of Ohio (Pavey, et, al, 1999) The geology at the project site is mapped as a late Wisconsinan-age ice-deposited soils of end moraine that occur as hummocky ridges higher than adjacent terrain.

Based on the Bedrock Geologic Units Map of Ohio (USGS & ODGS, 2006), bedrock within the project area consists of shale and limestone, of the Drakes, Whitewater, and Liberty formations, Undivided. This unit is comprised of Ordovician-age interbedded shale, and limestone. The interbedded shale and limestone are described as gray to maroon and weathers yellowish gray, planar to irregular to wavy, and thin to thick bedded. Bedrock rises gently from north to south (ODGS, 2003). Based on the ODNR bedrock topography map of Ohio, bedrock elevations at the project site can be expected to be between about 900 and 950 ft. amsl, putting bedrock at a depth ranging from about 40 ft below ground surface (bgs) to about 75 ft. below ground surface (bgs).

The soils at the project site have been mapped (Web Soil Survey) by the Natural Resources Conservation Service (USDA, 2015) as primarily Udorthents. Udorthents are soils that have been disturbed by large amounts of cutting and filling and as such are not rated according to the AASHTO method of soil classification. The soils surrounding the project site are mapped as primarily Miamian silt loam or clay loam and are characterized as very deep, well drained soils that are moderately deep or deep to dense till formed in loess and the underlying loamy till on till plains and moraines. The Miamian series is comprised of primarily fine-grained soils and classifies as cohesive A-4, A-6, and A-7 type soils according to the AASHTO method of soil classification.

According to the Water Well Log (ID# 2040231) groundwater at the project site can be expected at an elevation of about 6 ft bgs in the vicinity of the project's boundaries. ODOT's Office of Geotechnical Engineering has determined that a groundwater level of 945 feet should be used in the analyses for RW-3, which we have complied with. However, it should be noted that perched groundwater systems may be existent in areas due to the presence of fine-grained soils making it difficult for groundwater to permeate to the phreatic surface.

The project site is not located within a flood hazard area based on available mapping by the Federal Emergency Management Agency's (FEMA) National Flood Hazard mapping program (FEMA, 2016).

3.0 EXPLORATION

The following report/plans were available for review and evaluation for this report:

- Project Boring Logs for Structure Foundation Investigation for Project MOT-75-06.035 dated October 23, 1995.
- Project Boring Logs from Geological Report for Project MOT-725-14.10 dated October, 1976.
- Project Boring Logs from Geological Report for Project MOT-25-0374 dated August 4, 1958.

Historical soil borings associated with the above plans were reviewed. The subsurface exploration in Appendix A includes the historical borings that were selected to be used to determine the soil profiles/parameters for the design of the project retaining wall.

3.1 FIELD EXPLORATION

The exploration for all 4 walls was conducted by National Engineering & Architectural Services (NEAS) between February 23, 2022, and March 10, 2022 and included 24 borings drilled to depths between 7.5 ft to 26.5 ft bgs. The boring locations were selected by NEAS in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions. Borings were typically located along/near the proposed wall alignment in locations that were not restricted by maintenance of traffic, underground utilities or dictated by terrain (i.e., steep embankment slopes). Each as-drilled project boring location and corresponding ground surface elevation was surveyed in the field by NEAS following drilling. Each individual project boring log (included within Appendix A) includes the recorded boring latitude and longitude location (based on the surveyed Ohio State Plane South, NAD83, location) and the corresponding ground surface elevation. The boring locations are depicted on the boring plans provided in Appendix A.

Borings were drilled using a CME 45B truck mounted drilling rig utilizing 3.25-inch diameter hollow stem augers. Soil samples were recovered at intervals of 2.5-ft to end of boring using a split spoon sampler (AASHTO T-206 “Standard Method for Penetration Test and Split Barrel Sampling of Soils.”). The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist and possible laboratory testing. Standard penetration tests (SPT) were conducted using a CME auto hammer that has been calibrated to be 72.6% efficient as indicated on the boring logs on January 24, 2022.

Field boring logs were prepared by drilling personnel, and included lithological description, SPT results recorded as blows per 6-inch increment of penetration and estimated unconfined shear strength values on specimens exhibiting cohesion (using a hand-penetrometer). Groundwater level observations were recorded both during and after the completion of drilling. These groundwater level observations are included on the individual boring logs. After completing the borings, the boreholes were backfilled with either auger cuttings, bentonite chips, or a combination of these materials.

The following current test borings were utilized to define the soil profile for Retaining Wall 3: B-006-0-21, B-007-0-21, B-008-0-21, and B-009-0-21. In addition, historic borings B-001-0-95 and B-002-0-95 from the 1995 ODOT Structure Foundation Investigation were also utilized.

3.2 LABORATORY TESTING PROGRAM

The laboratory testing program consisted of classification testing and moisture content determinations. Data from the laboratory-testing program were incorporated onto the boring logs.

3.2.1 Classification Testing

Representative soil samples were selected for index properties (Atterberg Limits) and gradation testing for classification purposes on approximately 36% of the soil samples obtained. At each boring location, samples were selected for testing with the intent of identification and classification of all significant soil units. Soils not selected for testing were compared to laboratory tested samples/strata and classified visually. Moisture content testing was conducted on all samples. The laboratory testing was performed in general accordance with applicable AASHTO specifications.

A final classification of the soil strata was made in accordance with AASHTO M-145 “Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes,” as modified by ODOT “Classification of Soils” once laboratory test results became available.

3.3 STANDARD PENETRATION TEST RESULTS

Standard Penetration Tests (SPT) and split-barrel (commonly known as split-spoon) sampling of soils were performed at varying intervals (i.e., 2.5-ft or 5.0-ft intervals) in the project borings. To account for the high efficiency (automatic) hammers used during SPT sampling, field SPT N-values were converted based on the calibrated efficiency (energy ratio) of the specific drill rig's hammer. Field N-values were converted to equivalent rod energy of 60% (N60) for use in analysis or for correlation purposes. The resulting N60 values are presented on the boring logs provided in Appendix A. The hammer efficiency rating for the two 1995 ODOT borings was assumed to be 80%. To establish more conservative soil parameters, the values of N60 calculated for the two 1995 borings were rounded down prior to developing the soil unit weights, strength and bond strengths from the guidance literature. The N60 values shown in the logs for the four 2022 borings were checked by calculation and were also rounded down for parameter estimation.

4.0 FINDINGS

The subsurface profile within the proposed project area generally consists of surficial materials comprised of asphalt and base, generally underlain by natural stiff to hard cohesive soils and loose to dense granular soils. The natural stiff to hard cohesive soils encountered at the site of retaining walls consists of Sandy Silt (A-4a), Silt (A-4b) and Silt and Clay (A-6a). The loose to dense granular soils consists of Sandy Silt (A- 4a), Silt (A-4b), Course and Fine Sand (A-3a), Gravel and Stone Fragments with Sand (A-1-b), Stone Fragments with Sand and Silt (A-2-4) and Stone Fragments with Sand, Silt and Clay (A-2-6). Bedrock was only encountered in the historical borings near RW 1.

At the proposed RW 3 site, four project borings (B-006-0-21 to B-009-0-21) were drilled and indicate that the subsurface profile at the RW 3 site is very consistent. Bedrock was not encountered in any of the four project borings. Two soil strata were encountered and intersected with each other. The cohesive soils were classified on the project boring logs as Sandy Silt (A-4a), Silt (A-4b) and Silt and Clay (A-6a). Those cohesive soils can be described as stiff to hard consistency correlating to converted SPT-N values (N60) between 6 and 68 bpf. Natural moisture contents ranged from 10% to 15%. Based on Atterberg Limits test performed on representative samples of this material, the liquid limit is between 19 to 25 percent and plastic limit is between 13 to 14 percent. The granular soil stratum was only encountered at the beginning of wall (B-006-0-21 and B-007-0-21) and consisted of Gravel and Stone Fragments with Sand (A-1-b), Gravel with Sand and Silt (A-2-4) and Sandy Silt (A-4a). The granular soils can be described as loose to dense compactness correlating to converted SPT-N values (N60) between 9 and 42 bpf. Natural moisture contents ranged from 5% to 17%. It should be noted that boulder zone was encountered on the boring B-009-0-21 from 22.5 ft bgs to end of boring (from the elevation of 939.2 ft to 936.2 ft).

Groundwater measurements were taken during the boring drilling procedures and immediately following the completion of each borehole. Groundwater was not observed during drilling and upon completion in any of the structure borings performed as part of the referenced project.

It should be noted that groundwater is affected by many hydrologic characteristics in the area and may vary from those measured at the time of the exploration.

5.0 ANALYSES AND RECOMMENDATIONS

5.1 GENERALIZED SOIL PARAMETERS

Project borings B-006-0-21, B-007-0-21, B-008-0-21, and B-009-0-21 along with historic borings B-1 and B-2 from the 1995 ODOT metric Structure Foundation Investigation, MOT-75-6.03 were used to develop the soil profile and determine soil parameters. Each boring log was reviewed, and a generalized material profile was developed for analysis purposes. Utilizing the generalized soil profile, engineering properties for each soil strata were estimated based on the field and laboratory test results. ODOT's Geotechnical Design Manual (GDM) Sections 404 and 405 were utilized to estimate unit weights and strengths. Geotechnical Engineering Circular 7 (GEC 7) Tables 4.4a and b, and Table 4.6 were used to estimate the soil Estimated Bond Strengths. The developed soil profile and estimated engineering soil properties are summarized within the Tables 5-1 to 5-6 below. Note that as there was a range of N60 values throughout any specific layer, a range of values is provided.

Table 5-1 Retaining Wall: Soil Profile, B-001-0-95

Soil Description	Range						
	Total Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Effective Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(2) (psf)	Effective Friction Angle(3) (degrees)	Estimated Bond Strength(4) (psi)
Sandy Silt Elevation (977.5 ft - 968.7 ft)	125 - 130	115 - 120	63 - 67	3125 - 4500	260 - 330	35 - 38	9 - 22
Sandy Silt Elevation (968.7 ft - 966.2 ft)	132	122	70	5,300	370	35	9 - 22
Sandy Silt Elevation (966.2 ft - 935 ft)	128 - 140	118 - 130	66 - 78	3600 - 7500	290 - 470	33 - 35	9 - 22
Silt and Clay Elevation (935 ft - 922.5 ft)	132 - 140	122 - 130	70 - 78	5100 - 7500	360 - 470	34 - 35	5 - 7

Notes:

1. *Values interpreted from Geotechnical Design Manual Section 405.*
2. *Values interpreted from Geotechnical Design Manual Section 404.1.*
3. *Values interpreted from Geotechnical Design Manual Section 404.2.*
4. *Values interpreted from Geotechnical Engineering Circular 7, Tables 4.4a, 4.4b, and 4.6*

Table 5-2 Retaining Wall: Soil Profile, B-002-0-95							
Soil Description	Range						
	Total Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Effective Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(2) (psf)	Effective Friction Angle(2) (degrees)	Estimated Bond Strength(4) (psi)
Sandy Silty Clay Elevation (975.5 ft - 974 ft)	125	115	63	2500 - 3100	240	35	5 - 7
Sandy Silt Elevation (974 ft - 956.7 ft)	125 - 140	115 - 130	63 - 78	3100 - 6700	250 - 440	33 - 38	9 - 22
Silty Sand Elevation (956.7 ft - 953 ft)	132	122	70	-	-	37	15 - 22
Sandy Silt and Clay Elevation (953 ft - 920.5 ft)	125 - 140	115 - 130	63 - 78	3000 - 8000	250 - 500	31 - 35	9 - 22

Notes:
1. *Values interpreted from Geotechnical Design Manual Section 405.*
2. *Values interpreted from Geotechnical Design Manual Section 404.1.*
3. *Values interpreted from Geotechnical Design Manual Section 404.2.*
4. *Values interpreted from Geotechnical Engineering Circular 7, Tables 4.4a, 4.4b, and 4.6*

Table 5-3 Retaining Wall: Soil Profile, B-006-0-21							
Soil Description	Range						
	Total Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Effective Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(2) (psf)	Effective Friction Angle(3) (degrees)	Estimated Bond Strength(4) (psi)
Gravel with Sand Elevation (952.5 ft - 945.5 ft)	125 - 132	115 - 122	63 - 70	-	-	39 - 42	15 - 26
Sandy Silt Elevation (945.5 ft - 926 ft)	125 - 128	115 - 118	63 - 66	3000 - 3900	250 - 300	25	9 - 22

Notes:
1. *Values interpreted from Geotechnical Design Manual Section 405.*
2. *Values interpreted from Geotechnical Design Manual Section 404.1.*
3. *Values interpreted from Geotechnical Design Manual Section 404.2.*
4. *Values interpreted from Geotechnical Engineering Circular 7, Tables 4.4a, 4.4b, and 4.6*

Table 5-4 Retaining Wall: Soil Profile, B-007-0-21							
Soil Description	**Range**						
	Total Unit Weight(1) (pcf)	**Moist Unit Weight(1) (pcf)**	**Effective Unit Weight(1) (pcf)**	**Undrained Shear Strength(2) (psf)**	**Effective Cohesion(2) (psf)**	**Effective Friction Angle(3) (degrees)**	**Estimated Bond Strength(4) (psi)**
Gravel with Sand and Silt Elevation (953.7 ft - 946.7 ft)	118 - 125	108 - 115	56 - 63	-	-	35 - 38	15 - 22
Sandy Silt Elevation (946.7 ft - 941.7 ft)	125 - 128	115 - 118	63 - 66	2600 - 3300	230 - 275	33 - 35	9 - 22
Sandy Silt Elevation (941.7 ft - 939.2 ft)	125	115	63			33	9 - 22
Silt Elevation (939.2 ft - 929.2 ft)	125 - 128	115 - 118	63 - 66	3250 - 3750	270 - 295	33 - 35	9 - 11
Sandy Silt Elevation (929.2 ft - 927.2 ft)	125	115	63	3,600	290	33	15 - 22

Notes:
1. *Values interpreted from Geotechnical Design Manual Section 405.*
2. *Values interpreted from Geotechnical Design Manual Section 404.1.*
3. *Values interpreted from Geotechnical Design Manual Section 404.2.*
4. *Values interpreted from Geotechnical Engineering Circular 7, Tables 4.4a, 4.4b, and 4.6*

Table 5-5 Retaining Wall: Soil Profile, B-008-0-21							
Soil Description	**Range**						
	Total Unit Weight(1) (pcf)	**Moist Unit Weight(1) (pcf)**	**Effective Unit Weight(1) (pcf)**	**Undrained Shear Strength(2) (psf)**	**Effective Cohesion(2) (psf)**	**Effective Friction Angle(3) (degrees)**	**Estimated Bond Strength(4) (psi)**
Sandy Silt Elevation (958.1 ft - 943.6 ft)	115 - 122	105 - 112	52 - 60	750 - 2250	75 - 215	29 - 32	9-22
Sandy Silt Elevation (943.6 ft - 936.1 ft)	125 - 128	115 - 118	63 - 66	2600 - 3870	230 - 300	32 - 34	9 - 22
Sandy Silt Elevation (936.1 ft - 931.6 ft)	135	125	73	5600 - 7500	380 - 470	35 - 38	9 - 22

Notes:
1. *Values interpreted from Geotechnical Design Manual Section 405.*
2. *Values interpreted from Geotechnical Design Manual Section 404.1.*
3. *Values interpreted from Geotechnical Design Manual Section 404.2.*
4. *Values interpreted from Geotechnical Engineering Circular 7, Tables 4.4a, 4.4b, and 4.6*

Table 5-6 Retaining Wall: Soil Profile, B-009-0-21							
Soil Description	**Range**						
	Total Unit Weight(1) (pcf)	**Moist Unit Weight(1) (pcf)**	**Effective Unit Weight(1) (pcf)**	**Undrained Shear Strength(2) (psf)**	**Effective Cohesion(2) (psf)**	**Effective Friction Angle(3) (degrees)**	**Estimated Bond Strength(4) (psi)**
Silt and Clay Elevation (961.7 ft - 944.7 ft)	118 - 128	108 - 118	56 - 66	1125 - 3750	113 - 290	30 - 35	8.5 - 15
Silt and Clay Elevation (944.7 ft - 939.2 ft)	125 - 128	115 - 118	63 - 66	3250 - 3625	280	33 - 34	8.5 - 15

Notes:
1. *Values interpreted from Geotechnical Design Manual Section 405.*
2. *Values interpreted from Geotechnical Design Manual Section 404.1.*
3. *Values interpreted from Geotechnical Design Manual Section 404.2.*
4. *Values interpreted from Geotechnical Engineering Circular 7, Tables 4.4a, 4.4b, and 4.6*

When utilizing Table 4.6 in GEC 7, the rotary drill method was used, and a 6-inch diameter drill hole was assumed. In most cases, the values for estimated bond strength from Tables 4.4a and 4.4b were more conservative than those estimated from Table 4.6. The more conservative value is included in the tables above.

Summaries of the evaluations for the estimation of the unit weights and strength parameters are included in Appendix A.

5.2 RETAINING WALL 3

5.2.1 Cross-Section Development

Retaining Wall 3 is 450.0 feet long and extends from SR 725 WB BL Sta. 461+45.18, 34.5' Rt. to Sta. 466+02.72, 25.11' Rt. (Wall Stas. 3500+00 to 3504+50.00). It is located approximately 40 feet to the right of the SR 725 CL. Wall 3 is to be a soil nail wall that starts to the west of the IR 75 SBL bridge over SR 725, traverses under IR 75, and finishes to the east of the NBL bridge over SR 725. The wall will create room for the pedestrian path running along the south side of SR 725. The general design is for nails to be installed at the spacing required, with shotcrete facing applied to exposed slope surfaces, and an approximately 8-inch thick reinforced concrete facing over the nails and shotcrete. The shotcrete and concrete facing should extend to 4-feet below the proposed grade at the front of the wall, where the concrete facing will be founded on a small strip footing.

The preliminary design height for the wall is determined by the elevation of the slope intersection with the back of the wall down to the proposed groundline at the face of the wall. The maximum preliminary height of the wall is approximately 9.2 feet occurring near Wall Sta. 3501+15. However, considering the 4-foot extension of the wall facing below grade, and assuming little to no passive resistance for the sidewalk and fill in front of the wall, the design wall heights are assumed to be 4-feet taller than the preliminary height. This makes for a 13.2 foot high wall design near Sta. 3501+15.

Based on changes in the soil stratigraphy as well as the slope geometry, wall height, vertical and horizontal spacing of the soil nails, and live load surcharge, 5 soil profiles were developed to analyze/design the wall utilizing Snail v. 2.2.2 software. The following stations and wall layout were used in the section development:

- Stations 3500+00 to 3502+60. Station analyzed 3501+13. Wall height 13.2. Nail spacing 5 ft. horizontal, 3 ft. vertical. 4 rows of nails.
- Stations 3502+60 to 3503+05. Station analyzed 3502+85. Wall height 11.7. Nail spacing 5.25 ft. horizontal, 2.25 ft. vertical. 4 rows of nails.
- Stations 3503+05 to 3503+30. Station analyzed 3503+15. Wall height 11.1. Nail spacing 5 ft. horizontal, 3 ft. vertical. 3 rows of nails.
- Stations 3503+30 to 3504+45. Station analyzed 3503+70. Wall height 11.4. Nail spacing 4.5 ft. horizontal, 3 ft. vertical. 3 rows of nails.
- Stations 3504+45 to 3504+50. Station analyzed 3504+47. Wall height 8.5. Nail spacing 2.3 ft. horizontal, 4.3 ft. vertical. 2 rows of nails.

The following cross-sections (Figs. 2 to 6) represent the 5 locations to be analyzed. The soil profile and parameters indicated are based on the interpolation between borings near the station analyzed, a conservative approach to estimating soil parameters, and engineering judgement. Concerning soil nail analyses, the profiles were only developed to a depth where the lowest nails would extend, assuming a nail length of 20-feet at a 15° angle below horizontal. The water table at EL. 945 was not indicated on the cross-sections as the nail depths did not reach that elevation.

In order to compensate for soil profile variation through the wall length, the bond strength for all soils was assumed to be the lowest value estimated throughout the wall profile, which was 8.5 psi. The unit weight and strength parameters used in calculations for the nails and global stability may differ slightly than indicated in the profiles as engineering judgement was utilized to develop the best designs.

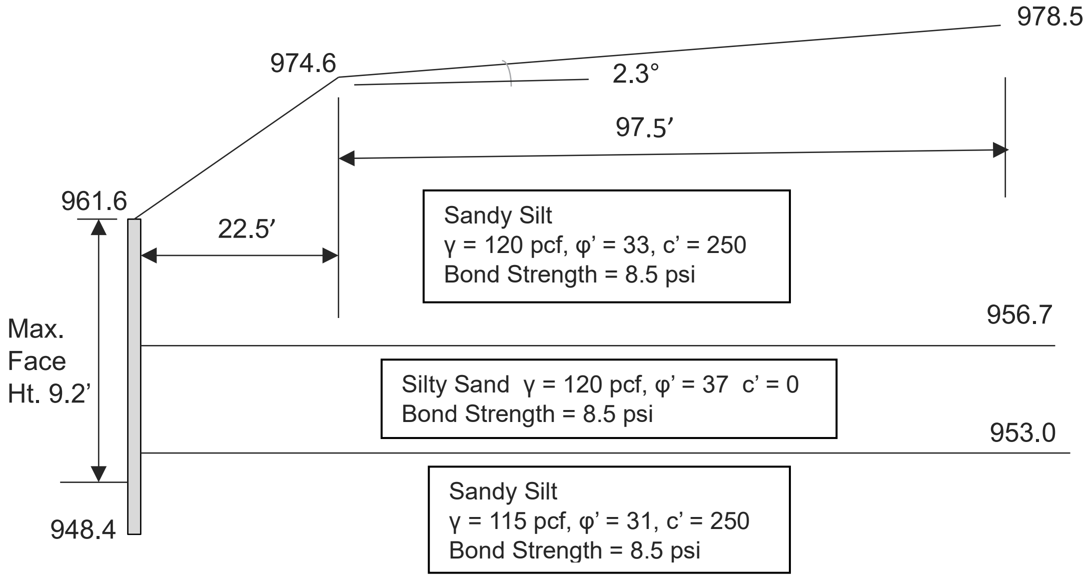

Fig. 2 Station 3501+13 Cross-section NTS

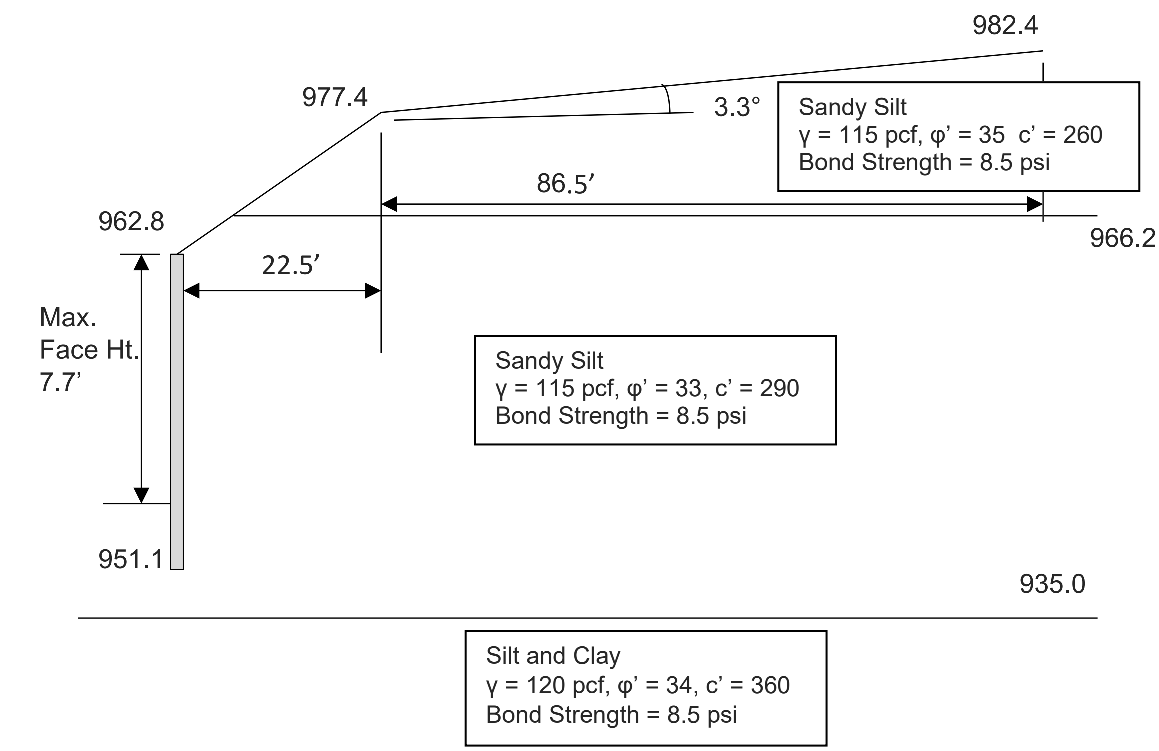

Fig. 3 Station 3502+85 Cross-section NTS

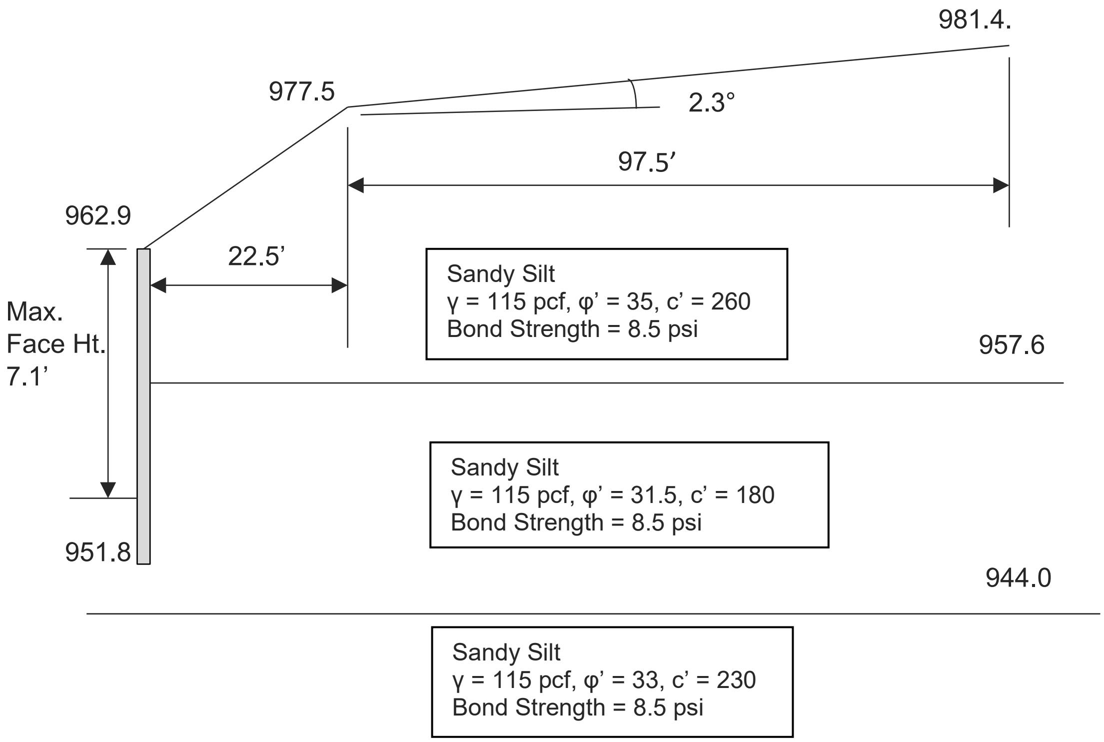

Fig. 4 Station 3503+15 Cross-Section NTS

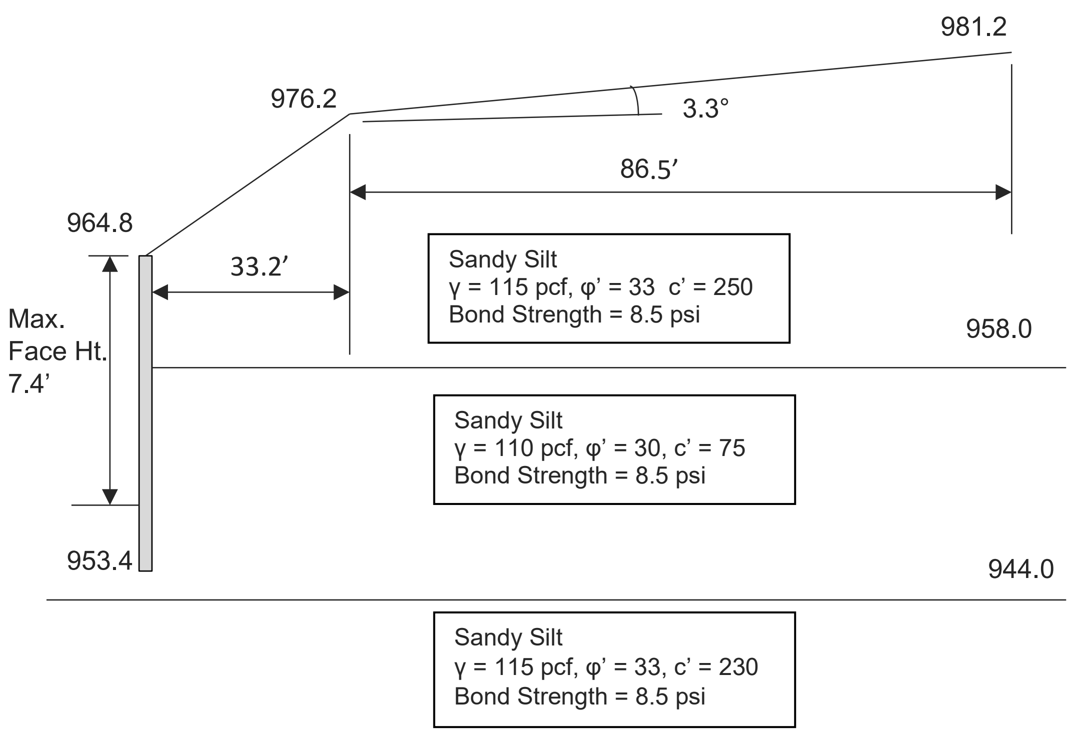

Fig. 5 Station 3503+70 Cross-section NTS

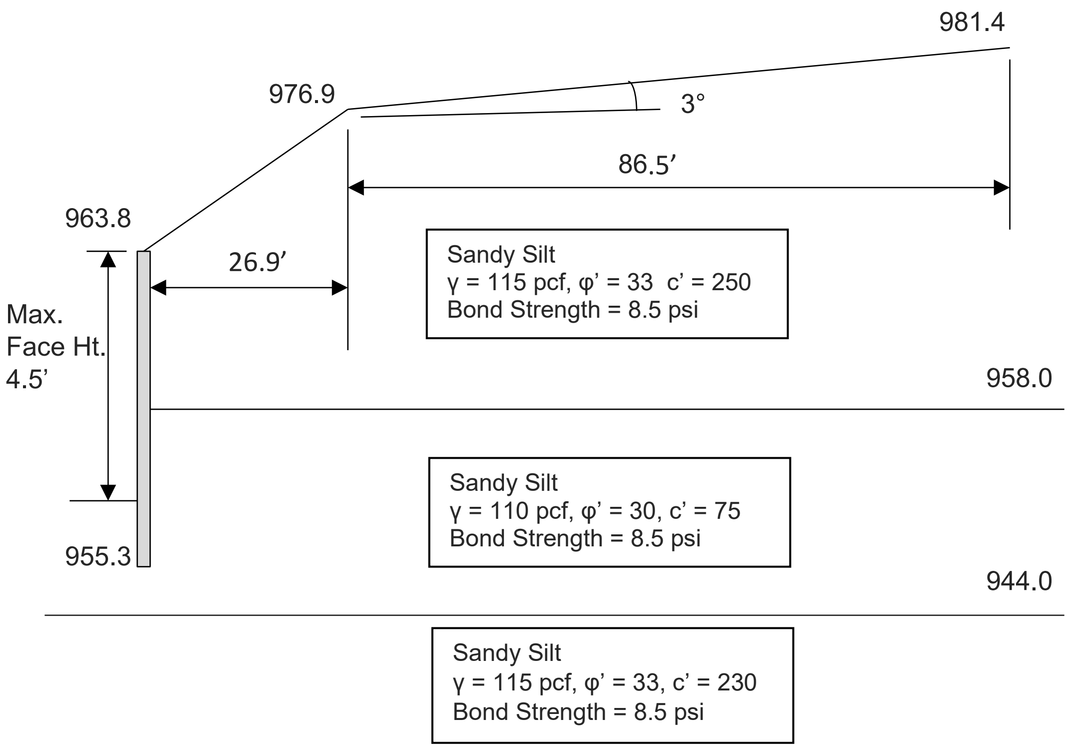

Fig. 6 Station 3504+47 Cross-section NTS

5.2.2 Soil Nail Wall Design

5.2.2.1 Soil Nail Design

Wall 3 was designed following Section 11.12 of the *AASHTO LRFD Bridge Design Specifications*. A surcharge of 250 psf was utilized for the wall sections below IR 75. However, to establish consistent and conservative designs, the 250 psf loading was applied at the top of all slopes behind the walls as well. Computer software Snail v. 2.2.2 by Caltrans was utilized to determine the optimal soil nail pattern consisting of nail lengths, diameters, steel yield strength, vertical and horizontal spacing, and angle of inclination. The software evaluates various failure geometries, and the results of the analyses provide the point of minimum capacity/demand ratio from which the Factored Design Loads (FDLs) were derived. The minimum capacity/demand ratio must be greater than 1.0 for a satisfactory design. FDLs are estimated as the stress on the nail (ksi) multiplied by the cross-sectional area of the steel bar. Using a No. 8 bar, diameter of 1-inch, yields an area of 0.79 square inches. The maximum stress in any of the nails at the critical search point (minimum capacity/demand ratio location) was used as the stress for all the nails in the stations analyzed. The software also performed checks on the reinforced concrete facing.

The standard nail spacing is 4.5 feet laterally and 3 feet or less vertically. The soil nails at some locations will need to go between the existing piles at the bridge abutments. The location of each soil nail was individually determined near the bridge structures. In some cases the angle of inclination of the top row of nails was changed from 15 degrees to 12 degrees to avoid installation interference from the bottom of the pier cap. The nail column stationing was shifted at some locations to miss the abutment piles or the pier columns. At many nail locations, a slight horizontal angle was utilized to achieve the necessary clearance from the piles.

The five sections analyzed were performed to check that there would not be a failure in locations where the spacing had to be increased. In case the Contractor encounters a boulder, debris, or an abutment pile in a different location than indicated on the historic plans, and it is necessary to move a soil nail, then guidance should be given in the plans as to the maximum allowable horizontal and vertical spacing and the total number of nails required between expansion and contraction joints.

In addition to the initial 5 sections analyzed, the critical section at Sta. 3501+13 was analyzed for the areas where the top nails are inclined at 12 degrees instead of at 15 degrees. Another extra analysis was carried out with groundwater included at elevation 946.6. Groundwater was shown at this elevation because the model was developed at station 3501+13 in which the bottom nail is 1.6 feet higher than nail one in the very first nail column. Therefore, to mimic the worst possible conditions in which the groundwater table may have the most impact on the soil nail wall, the groundwater table was artificially shown to be 1.6 feet higher.

All 7 critical sections were evaluated in Snail with 20-foot long nails generally at 15 degrees from the horizontal. The nails were 60 ksi steel No. 8 bars with 6-inch drill holes, spaced as laid out for the given sections. The facing was input as an 8-inch thick, 4000 psi., reinforced concrete facing, with 9-inch square bearing plates. All soils were given a bond strength of 8.5 psi. There will be a very short time of temporary conditions, but Snail utilizes the soil's long term effective strengths. The effective cohesion and friction angles were input. LRFD load and resistance factors for the soil nails permanent condition were input as follows:

Soil Nails	
LRFD Values	**Factor**
Load	
Soil Nail Tensile Force	1.35
Resistance	
Pullout (Distal)	0.65
Pullout (Proximal)	0.65
Nail Bar Yield	0.75
Cohesion	1.0
Friction Angle	1.0

Table 5-7 indicates the 7 stations analyzed, the minimum capacity/demand ratio, nail stress, and the FDL for each of those sections. The Snail input and output diagrams, as well as the input/results output in tabular form for the nails, are included in Appendix B.

Table 5-7 Factored Design Loads

Station Analyzed	Wall Stations	Top of Wall Elevation	Design Wall Height ft	Between Nail Spacing	Nail Rows and Inclination Angle	Minimum Capacity/ Demand Ratio	Nail Stress ksi	FDL kips/nail
3501+13	3500+00 to 3502+60	961.6	13.2	5' H 3' V	4 rows @ 15°	1.87	20.4*	16.2
3501+13 Water Table	3500+00 to 3502+60	961.6	13.2	5' H 3' V	4 rows @ 15°	1.87	20.4*	16.2
3501+13 Top Nail @ 12°	3500+00 to 3502+60	961.6	13.2	5' H 3' V	Bottom 3 rows @ 15° Top row @ 12°	1.87	20.4*	16.2
3502+85	3502+60 to 3503+05	962.8	11.7	5.25' H 2.25' V	4 rows @ 15°	2.18	19.8	15.6
3503+15	3503+05 to 3503+30	962.9	11.1	5' H 3' V	3 rows @ 15°	2.00	20.7	16.4
3503+70	3503+30 to 3504+45	964.8	11.4	4.5' H 3' V	3 rows @ 15°	2.48	14.9	11.8
3504+47.8	3504+45 to 3504+50	963.8	8.5	2.3' H 4.3' V	2 rows @ 15°	2.71	14.9	11.8

* These models were also evaluated with no grade beyond the slope crest, yielding a more conservative stress (20.4 ksi) as opposed to with a grade beyond the crest (19.8 ksi).

On the RW 3 plans included in Appendix B, all the nails have been numbered sequentially from 1 to 363, starting at the lower row and proceeding west to east. In Table 5-8, the design FDL (rounded up) and angle from the horizontal for each section of nails is provided utilizing this numbering. To minimize FDL variations, the nails from Sta. 3502+60 to 3503+05 should be constructed for an FDL of 17 kips/nail to match the surrounding sections. The values in Table 5-8 should be included in the plans for the Contractor to determine their nail construction procedure.

Table 5-8 Design Nail Plan Values		
Nail Numbers	**Design FDL Kips/nail**	**Angle Down From Horizontal**
1 - 75 104 - 178 205 - 278 289 - 335	17.0	15°
279 - 288	17.0	12°
76 – 103 179 – 204 336 - 363	12.0	15°

At certain locations the horizontal angle for specific nails has been evaluated so that the nails will miss the piles under the abutment. The specific nails and the angle and direction required for them to go between the piles are shown in Table 5-9. Any nail not indicated in the table is at 0 degrees off the perpendicular line from the wall face.

Table 5-9 Nail angles from Perpendicular	
Nail Numbers	**Angle from Perpendicular °**
13, 116, 213, 273	2.5° to the east
23, 61, 71, 72, 76, 126, 164, 174, 175, 179, 223, 261, 283, 321, 331, 332, 336	5° to the east
16, 20, 21, 28, 33, 64, 79, 119, 123, 124, 131, 136, 167, 182, 216, 220, 221, 228, 233, 264, 276 280, 281, 288, 293, 324, 339	5° to the west

As noted earlier, the standard nail spacing is 4.5 feet laterally and 3 feet or less vertically. On the east end of the wall, the vertical spacing is greater than 3.5 feet between nails 102 and 362, and between nails 103 and 363. Vertical and horizontal spacing at other locations will often differ than the standard spacing. It is more constructive for the various spacings to be shown on the plans as opposed to developing a report table. The geotechnical engineer should be intimately involved with the plan development in order to ensure that the correct spacing is indicated on the RW 3 plans. Special angle plates will need to be developed to compensate for the horizontal installation angle. The various nail horizontal and vertical spacings have been indicated on the RW 3 plans included in Appendix B.

It should be noted that GEC 7 Section 6.3.3b discusses soil nail spacing with regards to the depth of the top row and the height of the bottom row relative to the top and bottom of the wall, respectively. The following paragraph is from this section:

"The first row of nails should not be installed deeper than approximately 2 to 3.5 ft from the top edge of the wall to reduce the potential for instability of the upper excavation lift and to reduce cantilever effects on the temporary facing. The lowermost row of nails should be installed about 2 to 3 ft above the base of the excavation. These requirements are the result of the limited ability of the facing to work as a cantilever at the top and bottom of the wall. However, these limits may be adjusted for project-specific conditions, and when based on suitable analysis."

There are locations where the nails installed at RW 3 will be slightly outside these GEC 7 recommended values. Taking into account the proposed ditch behind the wall, the 4 ft of buried wall section, and the required spacing between nails, the upper and lower rows of nails at some locations will be less than 2.0 ft below the top of wall or less than 2.0 ft above the bottom of the wall. These alterations from the GEC 7 recommendations were considered necessary and the appropriate analyses with the nail locations showed adequate nail responses.

5.2.2.2 Facing Design

The proposed facing along the wall was evaluated in Snail, and was input as an 8-inch thick, 4000 psi., reinforced concrete facing, with 9-inch square bearing plates. Four shear studs were assumed per plate. Only permanent conditions were considered. The following LRFD factors were included in the evaluation:

Facing	
LRFD Values	**Factor**
Facing Resistance	
Punching	0.9
Flexural	0.9
Stud Tensile	0.7
Bearing Plate Resistance	
Tensile Stress	0.68
Flexural	0.84
Bearing Stress of Steel	1.08
Bearing Stress of Concrete/Shotcrete	0.41

Facing evaluation in Snail develops a resistance/load ratio, which equals the factored resistance of the facing divided by the empirical factored load at the nail head. A ratio of greater than 1 indicates acceptable resistance. Table 5-10 provides the facing analyses results for each of the seven station sections evaluated. All resistance/load ratios are greater than 1.0 and the facing design as analyzed is satisfactory. The results from the facing design performed by Snail is included in Appendix B.

Table 5-10 Facing - Permanent Factored Loads and Capacity Ratios				
Station Analyzed	Wall Stations	Factored Load at Nail Head kips	Factored Facing Resistance kips	Resistance Load Ratio
3501+13	3500+00 to 3502+60	15.4	42.0	2.7
3501+13 Water Table	3500+00 to 3502+60	15.4	42.0	2.7
3501+13 Top Nail @ 12°	3500+00 to 3502+60	15.4	42.0	2.7
3502+85	3502+60 to 3503+05	15.3	42.0	2.7
3503+15	3503+05 to 3503+30	15.6	42.0	2.6
3503+70	3503+30 to 3504+45	10.8	42.0	3.8
3504+47.8	3504+45 to 3504+50	10.8	42.0	3.8

5.2.2.3 Global Stability

The evaluation of overall stability of the bridge slope with the soil nail wall was investigated at the service limit state as specified in Article 11.6.2.3 of AASHTO. Overall stability was evaluated by a limiting equilibrium method using modified Bishop. Article 11.6.2.3 recommends that overall stability be evaluated at the Service Limit State (i.e. a load factor of 1.0) and a resistance factor of 0.65 for slopes which support a structural element. Available slope stability programs produce a single factor of safety for overall global stability. Since the resistance factor is combined with a load factor of 1.0, the resistance factor of 0.65 is equivalent to a safety factor of 1.5 for slopes which support a structural element.

Global stability, both short term and long term, was evaluated at 5 wall sections using Slide2 software by Rocscience. To examine a section near Sta. 3501+13, section 3501+25 was utilized, with the water table, and this section was used to represent the 3 sections analyzed at Sta. 3501+13. The remaining 4 sections analyzed were the four other stations used in the soil nail calculations. Soil profiles and parameters were adjusted from the nail analyses in order to evaluate short and long term conditions. Table 5-11 shows the wall stationing and the FS for the short term and long term analyses.

Table 5-11 Global Stability Results			
Station Analyzed	**Wall Stations**	**Short Term FS**	**Long Term FS**
3501+25	3500+00 to 3502+60	5.5	2.1
3502+85	3502+60 to 3503+05	6.9	2.2
3503+15	3503+05 to 3503+30	2.7	1.8
3503+70	3503+30 to 3504+45	3.1	2.3
3504+47.8	3504+45 to 3504+50	3.2	2.1

All of the FS for global stability, both short and long term, are greater than 1.5, which is acceptable. Results of the stability analyses are included in Appendix B.

5.2.3 Construction Considerations, Plan Notes and Special Provisions

The ODOT Special Provisions ‘Soil Nail Retaining Wall’, issued September 13, 2021 is to be included as part of the construction plans. No modification to the Special Provision should be necessary.

Plan Note ‘Soil Nail Retaining Wall’, from the Office of Geotechnical Engineering at ODOT, shall be included in the plans. Following is a copy of the plan note to be used, with the project specific modifications included:

SOIL NAIL RETAINING WALL

DESCRIPTION OF WORK:

THIS WORK CONSISTS OF CONSTRUCTING A PERMANENT SOIL NAIL WALL AS SPECIFIED HEREIN, AS SHOWN ON THE CONTRACT DRAWINGS, AND PER THE PROJECT SPECIAL PROVISIONS. FURNISH ALL LABOR, MATERIALS, EQUIPMENT, AND INCIDENTALS TO COMPLETE THE WORK. DESIGN THE SOIL NAIL WALL TO MEET THE MINIMUM REQUIREMENTS SPECIFIED HEREIN, SHOWN ON THE CONTRACT DRAWINGS, OR SPECIFIED IN THE PROJECT SPECIAL PROVISIONS.

DESIGN:

REFERENCE: AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS ARTICLE 11.12, SOIL NAIL WALLS. THE BATTER OF THE DESIGNED WALL IS 0 DEGREES. PROVIDE 363 SOIL NAILS WITH A FACTORED DESIGN LOAD AS SHOWN IN THE SOIL NAIL FDL AND INCLINATION ANGLE TABLE. THE NAILS SHALL CONSIST OF A MINIMUM OF 60 KSI STEEL, WITH A MAXIMUM 3.5 -FOOT VERTICAL SPACING AND A MAXIMUM 5.5 -FOOT

*HORIZONTAL SPACING. THE NAILS SHALL BE INSTALLED AT AN INCLINATION OF __15 DEGREES FROM THE HORIZONTAL, EXCEPT AS NOTED IN THE SOIL NAIL FDL AND INCLINATION ANGLE TABLE. HOLLOW BAR SOIL NAILS (HBSN) ARE ALLOWED. THE SHOTCRETE FACING REINFORCEMENT SHALL CONSIST OF WELDED WIRE FABRIC PER ASTM A1060. THE SHOTCRETE WALL FACING SHALL BE PER ITEM 520, WITH A MINIMUM THICKNESS OF _4__". THE CAST-IN-PLACE PERMANENT WALL FACING SHALL BE PER ITEM 511, CLASS QC1 CONCRETE, WITH A MINIMUM THICKNESS OF _8__", AND SHALL BE REINFORCED PER THE WALL DETAILS. FOR ALL EVALUATIONS OF OVERALL STABILITY, THE DEPARTMENT CONSIDERS THE PROPOSED SOIL NAIL WALL TO BE A "CRITICAL" STRUCTURE AS DEFINED IN FHWA-NHI-14-007.***

WALL DRAINAGE SYSTEM:

PROVIDE ALL ELEMENTS OF THE SOIL NAIL WALL DRAINAGE SYSTEM CONSISTING OF GEOCOMPOSITE DRAIN STRIPS, PVC CONNECTION PIPE, AND WEEPHOLES, AS SHOWN IN THE CONTRACT DRAWINGS, THAT WILL PROVIDE A CONTINUOUS PATH FOR WATER FLOW AND PREVENT PORE WATER PRESSURE FROM BUILDING BEHIND THE WALL. PROVIDE GEOCOMPOSITE DRAIN STRIPS, WEEPHOLES, AND OUTLET PIPE PER ITEM 518.

TESTING:

PERFORM A MINIMUM OF _11__ SOIL NAIL VERIFICATION TESTS ON SACRIFICIAL PRE-PRODUCTION SOIL NAILS. PERFORM PROOF TESTS ON 5% OF THE SOIL NAILS IN EACH NAIL ROW OR A MINIMUM OF ONE PER ROW; AT LOCATIONS ACCEPTED BY THE ENGINEER.

BASIS OF PAYMENT:

THE FOLLOWING ESTIMATED QUANTITIES HAVE BEEN CARRIED TO THE GENERAL SUMMARY TO COMPLETE THE ABOVE WORK:
ITEM 530, SPECIAL - RETAINING WALL, SOIL NAIL, _363__, EACH
ITEM 530, SPECIAL - RETAINING WALL, SOIL NAIL VERIFICATION TEST, _11__, EACH
ITEM 530, SPECIAL - RETAINING WALL, SOIL NAIL PROOF TEST, _19__, EACH
ITEM 503, UNCLASSIFIED EXCAVATION, AS PER PLAN, LUMP SUM
ITEM 509, EPOXY COATED REINFORCING STEEL, AS PER PLAN, 34,963__ LB
ITEM 511, CLASS QC1 CONCRETE, RETAINING/WINGWALL NOT INCLUDING FOOTING, AS PER PLAN, _150_ CY
ITEM 518, PREFABRICATED GEOCOMPOSITE DRAIN, _400__ SY
ITEM 518, _3__ INCH CORRUGATED POLYETHYLENE SMOOTH LINED PIPE, INCLUDING SPECIALS, _170__ FT
ITEM 520, PNEUMATICALLY PLACED CONCRETE SHOTCRETE, AS PER PLAN, _5190__ SF

In addition to the above plan note, the following note should be included in the plans:

The estimated (minimum) soil nail length is 20 feet. The vertical spacing is greater than 3.5 feet between nails 102 and 362, and between nails 103 and 363.

The following table should be included as part of the Soil Nail Retaining Wall notes:

SOIL NAIL FDL AND INCLINATION ANGLE

NAILS	***FDL (KIPS/NAIL)***	***INCL. ANGLE (DEG.)***
1 - 75, 104 - 178, 205 - 278, 289 - 335	*17*	*15*
279 - 288	*17*	*12*
76 – 103, 179 – 204, 336 - 363	*12*	*15*

General Considerations

The biggest concern in the soil wall construction is the possible interference of the pier cap with installation of the upper row nails and the hitting of the abutment piles during drilling of the nails. The Contractor should strictly follow the angles and locations of the nails as shown in the plans. Should it happen that the pier cap will interfere with the installation, minor tweaks of the installation angle of installation can be made to avoid the pier. Any such modified installation will require that the nail be proof tested at no cost to the State. If the Contractor strikes a pile (or other obstructions) during nail installation, the Contractor shall stop the nail installation, fill the abandoned hole with grout, and relocate the nail as close as possible to the original plan location at no additional cost to the State.

The construction process should be carefully planned out. Exposing a considerable height of the slope face, for a long stretch of wall and prior to any nail installation, could be a source for shallow slope failures. Benching and width of excavation should be considered so as to not expose too long and too high a slope face prior to nail installation.

Due to most of the material behind the wall being non-plastic or having a low plasticity, the drill holes will be subject to collapsing. The Contractor can expect to use casings to keep drill holes open prior to nail insertion and grouting.

6.0 CONCLUSION

This report has been prepared to aid in the evaluation of this site and to assist designers with the design of Retaining Wall 3 for MOT-725-14.41, in Montgomery County, Ohio. The report's scope is limited to recommendations pertaining to a specific project and the location described. The project description represents our current understanding of the significant aspects of the proposed improvements that will be affected by geotechnical conditions. Should the contractor encounter water during the making of any excavations, the contractor should stop excavation and contact the design Engineer.

The analysis and recommendations contained in this report are based upon the data obtained from the test borings performed at the locations indicated on the boring location plan. This report does not reflect any variations which may occur between the borings. The nature and extent of the variations between borings or existing pavement cores may not become evident until the course of construction. If subsurface conditions different from those described are noted during construction, recommendations in this report must be re-evaluated.