Resource International, Inc.

FRA-70-22.85
RETAINING WALL 1
PID NO. 98232
FRANKLIN COUNTY, OHIO

STRUCTURE FOUNDATION EXPLORATION REPORT (REV. 1)

Prepared For:
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Rii Project No. W-17-140

May 2023

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December 22, 2022 (Revised May 6, 2023)

Mr. Steve Beal, P.E.
Senior Transportation Engineer
EMH&T
5500 New Albany Road
Columbus, Ohio 43054

Re: Structure Foundation Exploration (Rev. 1)
FRA-70-22.85 Far East Freeway
Retaining Wall 1
PID 98232
Franklin County, Ohio
Rii Project No. W-17-140

Mr. Beal:

Resource International, Inc. (Rii) is pleased to submit this revised Structure Foundation Exploration Report for the above referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of the proposed Retaining Wall 1 carrying westbound I-70 to northbound I-270, underneath the existing FRA-70-2293 bridge, as part of the FRA-70-22.85 project within the City of Columbus, in Franklin County, Ohio.

We sincerely appreciate the opportunity to be of continued service to you on this project. If you have any questions regarding the structure foundation exploration, or this report, please do not hesitate to contact us.

Sincerely,

RESOURCE INTERNATIONAL, INC.

Daniel E. Karch, P.E.
Project Manager – Geotechnical Services

Brian Trenner, P.E.
Vice President – Geotechnical Services

Enclosure: Structure Foundation Exploration Report (Rev. 1)

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EXECUTIVE SUMMARY

The overall purpose of this project is to provide detailed subsurface information and recommendations for the Phase 2 and 3 of the FRA-70-022.85 project. This report is a presentation of the structure foundation exploration performed for the proposed retaining wall along Ramp F carrying westbound I-70 to northbound I-270, underneath the existing FRA-70-2293 bridge. The proposed widening of Ramp F will extend into the existing spill through slope and requires a top-down (cut) retaining wall to support the soil in front of the forward abutment of the overhead bridge structure. The limits of the proposed retaining wall structure along the Ramp F baseline extend from station 1533+25 to station 1535+70, for a total length of approximately 245 feet. Due to the limited clearance beneath the existing structure, it is understood that a soil nail wall type is the preferred option.

Exploration and Findings

One (1) boring, identified as B-001-0-19, was performed for Wall 1 on August 4, 2020 to a depth of approximately 38.8 feet beneath the existing ground surface. The boring was performed along the embankment of the I-70 eastbound to I-270 northbound ramp. On November 30, 2021, one (1) additional boring, identified as B-001-8-21, was performed on the east side of the bridge abutment to a depth of 21.4 feet below existing grade. In addition to the borings performed for this project, Rii utilized historical boring B-011-0-67 in the area of the proposed Wall 1 structure available through the ODOT Transportation Information Mapping System (TIMS).

Underlying the existing fill material in boring B-001-0-19 and the surface material in boring B-001-8-21, the natural soils encountered consisted of both cohesive and granular deposits. The natural cohesive soils were described as silt and clay (ODOT A-6a). The natural granular soils were described as gravel with sand, silt and clay (ODOT A-2-6). It should also be noted that rock fragments were encountered in boring B-001-0-19 at depths ranging between approximately 21 feet and 30 feet below the existing ground surface.

Historic boring B-011-0-67 encountered cohesive and granular material identified as clayey sandy gravel (A-6b) and brown and gray sandy gravel and gravelly sand (ODOT A-1-b, A-2-4).

Bedrock was encountered in borings B-001-0-19 and B-001-8-21 at depths of 33.5 feet and 4.5 feet beneath the existing ground surface, or approximately elevation 769.9 feet and 775.4 feet msl. The bedrock was described as slightly to highly weathered black shale. Additionally, historic boring B-011-0-67 reported top of weathered rock (unclassified) was encountered at the completion depth of approximately 25 feet, or approximately elevation 771.

Groundwater was initially encountered during drilling in borings B-001-0-19 at a depth of 36.0 feet below the existing ground surface. Upon completion of drilling, measurable groundwater was observed in boring B-001-0-19 at a depth of 31.6 feet. Groundwater was not encountered in boring B-001-8-21 prior to the introduction of water for rock coring. Groundwater was not reported on the historic log for boring B-011-0-67.

Analyses and Recommendations

Soil Nail Wall Recommendations

The soil nail wall, Wall 1, was analyzed for internal stability using the Snail software analysis program developed by the California Department of Transportation. Based on plan information provided and the soil parameters selected, a vertical spacing of 3.0 feet and horizontal spacing of 5.0 feet was utilized in the analysis, and an inclination of 15° from horizontal was considered. An 8.0-inch diameter augered hole with a No. 8 steel reinforcement bar (1.0 in^2) with 60 ksi yield strength was considered for the soil nail cross section. A nail length of 30 feet was analyzed in order to meet stability requirements. The soil nail wall was evaluated for sliding stability, bearing capacity (basal heave), and global stability, and the results of the analyses indicate the proposed configuration is considered satisfactory for stability requirements.

Please note that this executive summary does not contain all the information presented in the report. The unabridged subsurface exploration report should be read in its entirety to obtain a more complete understanding of the information presented.

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the Phase 2 and 3 of the FRA-70-022.85 project. The project's proposed improvements include the reconfiguration of the north half of the Brice Road interchange and westbound ramps to Interstate 270 (I-270) interchange, replacement of the Brice Road Bridge over Interstate 70 (I-70), a proposed Brice Road Bridge over new WB-CD Ramp, three (3) noise barriers, twelve (12) retaining walls, and five (5) culvert extensions.

This report is a presentation of the structure foundation exploration performed for the proposed retaining wall along Ramp F carrying westbound I-70 to northbound I-270, underneath the existing FRA-70-2293 bridge. The proposed widening of Ramp F will extend into the existing spill through slope and requires a top-down (cut) retaining wall to support the soil in front of the forward abutment of the overhead bridge structure. The limits of the proposed retaining wall structure along the Ramp F baseline extend from station 1533+25 to station 1535+70, for a total length of approximately 245 feet. Due to the limited clearance beneath the existing structure, it is understood that a soil nail wall type is the preferred option.

The exploration was performed within general accordance of the Ohio Department of Transportation (ODOT) Specifications for Geotechnical Explorations (SGE), dated July 2020. The project site and general location of the proposed retaining walls are as shown on the vicinity map and boring plan presented in Appendix I.

2.0 RECONNAISSANCE AND PLANNING

2.1 Site Geology

Physiographically, the site lies within the Columbus Lowland District of the Southern Ohio Loamy Till Plain Region. This region is characterized by relatively flat-lying silty loam till ground moraine, interspersed with end and recessional moraines, outwash and alluvial deposits. Ground moraines are deposited during the retreat of a glacier, resulting in an undifferentiated mixture of clay, silt, sand and gravel. End moraines are normally associated with ice melting that is neither advancing nor retreating for a period of time. Recessional moraines are deposited when the ice sheet is retreating. Both end and recessional moraines are commonly associated with boulder belts. Outwash deposits consist of undifferentiated sand and gravel deposited by meltwater in front of glacial ice, and often occurs as valley terraces or low plains. Alluvium and alluvial terrace deposits range from silty clay to cobble sized deposits, usually deposited in present and former floodplain areas, such as the Big Walnut Creek and its tributaries.

Based on the Bedrock Geology and Bedrock Topography maps of the Columbus area, obtained from Ohio Department of Natural Resources (ODNR), the bedrock at the proposed project site consists of the Upper Devonian-aged Ohio Shale Formation. The Ohio Shale Formation is further subdivided into three primary members, in descending order: the Cleveland, Chagrin, and Huron Members. The Cleveland Member consists of black shale and is thickest in the north-central portion of the state but thins out to the south and east. The Huron Member consists of gray to greenish gray interbedded shale, siltstone, and very fine-grained sandstone, and is thickest in the northeastern portion of the state, thinning out to the southwest. The Chagrin Member grades into the overlying and underlying members and consists of black, carbonaceous shale. The entire Ohio Shale formation ranges from 250 to over 500 feet thick, with generally laminated to thin bedding and fissile partings, and is characterized by such features as having a petroliferous odor and carbonate/siderite concretions.

According to bedrock topography mapping from ODNR, the top of bedrock forms a ridge to the north of the site, generally lying just outside of the I-270 loop, and roughly underlying the cities of Gahanna and Reynoldsburg. The bedrock surface forms a narrow plateau that extends southwest from the south end of this ridge, which projects beneath the I-270 and I-70 interchange. The bedrock surface slopes down to the northwest and to the southeast from this plateau near the interchange, then generally slopes downward to the south and southeast. The bedrock near the interchange and northward along I-270 and eastward along I-70, lies at an approximate elevation of 750 feet mean sea level (msl), or approximately 27 to 33 feet below the ground surface. The bedrock surface gets only slightly deeper moving northward and approximately 50 feet deeper eastward from the interchange near the Brice Road overpass over I-70. The bedrock surface slopes upward moving northward along Brice Road from the Brice Road overpass over I-70.

2.2 Observations of the Project

The site of the proposed FRA-70-22.85 project is located along the east side of Columbus, in Franklin County, Ohio, with the project limits stretching from the east side approximately 1,400 feet east of the existing I-70 exit ramp to Brice Road, and extending westward along I-70 to the I-270 northbound ramp. On the north side, the project extends along Brice Road to the first intersection north of the bridge, and on the south side, the project extends along Brice Road to the intersection of Chantry Drive and Brice Road. Land use surrounding the majority of the project vicinity is predominantly commercial and residential units.

Based on the site reconnaissance of the project area in the vicinity of Wall 1, the existing pavement north of the forward abutment of the bridge over Ramp F appeared to be in fair condition with minor spalling/rutting near the expansion joint of the bridge approach. The existing embankment is approximately 20 feet in height at the forward abutment. The slopes of the embankment, with the exception of the spill through slope which was protected by aggregate, were heavily vegetated.

3.0 EXPLORATION

One (1) boring, identified as B-001-0-19, was performed for Wall 1 on August 4, 2020 to a depth of approximately 38.8 feet beneath the existing ground surface. The boring was performed along the embankment of the I-70 eastbound to I-270 northbound ramp. On November 30, 2021, one (1) additional boring, identified as B-001-8-21, was performed on the east side of the bridge abutment to a depth of 21.4 feet below existing grade. A summary of the borings analyzed for the subject structure is presented in Table 1.

Table 1. Summary of FRA-70-22.85 Wall 1 and 7 Borings

Wall ID	Boring Number	Alignment	Station	Offset	Latitude [1]	Longitude [1]	Ground Elevation (feet) [1]	Boring Depth (feet)
Wall 1	B-001-0-19	BL Ramp D2	1029+65	6.1 Lt.	39.934994	-82.848710	803.4	38.8
	B-001-8-21	BL Ramp D2	1028+81	17.2 Lt.	39.934761	-82.848719	779.9	21.4

1. Ground surface elevations and coordinates were provided by EMH&T survey.

Boring locations were determined and field located by Rii personnel prior to drilling operations. During the field locating and reconnaissance, Rii utilized a handheld GPS mark the boring locations. Coordinates and ground surface elevations of the as drilled boring locations were provided by the EMH&T survey team.

The borings performed for the subject structures were drilled with a truck-mounted rotary drilling machine, utilizing 3.25-inside diameter hollow-stem augers to advance the holes between sampling attempts. Standard penetration testing (SPT) and split spoon sampling were performed at 2.5-foot intervals to a depth of 25 feet below the existing ground surface and at 5.0-foot intervals thereafter to the boring termination depth in boring B-001-0-19 and to split spoon sampler refusal in boring B-001-8-21. Split spoon sampler refusal is defined as exceeding 50 blows with less than 6.0 inches of penetration by the split spoon sampler

The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted using a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outside diameter split spoon sampler 18.0 inches. Driving resistance is recorded on the boring logs in terms of blows per 6-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N_m) values are corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both values are represented on boring logs presented in Appendix III.

$$N_{60} = N_m*(ER/60)$$

Where:
N_m = measured N value
ER = drill rod energy ratio, expressed as a percent, for the system used

Borings B-001-0-19 was performed using the hammer for a truck-mounted CME-55 drill rig operated by Rii, which was calibrated on September 4, 2018, and has a drill rod energy ration of 91.2 percent. The energy ratio was limited to a maximum of 90 percent for evaluation of the SPT blow count data, in accordance with the ODOT SGE. Boring B-001-8-21 was performed using the hammer for a Mobile B-53 drill rig operated by Rii on this project that was calibrated on September 14, 2020, and has a drill rod energy ratio of 83.6 percent.

Hand penetrometer readings, which provide a rough estimate of the unconfined compression strength (UCS) of the soil, were reported on the boring logs in units of tons per square foot (tsf) and were utilized to classify the consistency of the cohesive soil in each layer. An indirect estimate of the unconfined compressive strength of the cohesive split spoon samples can also be made from a correlation with the blow counts (N_{60}). Please note that split spoon samples are considered to be disturbed and the laboratory determination of their shear strengths may vary from undisturbed conditions.

Following split spoon sampler refusal in boring B-001-8-21, rock coring was performed using a NQ-sized double-tube diamond bit core barrel (utilizing wire line equipment). Coring produced a 1.85-inch diameter core from which the type of rock and its geological characteristics were determined.

The rock cores obtained from the borings were logged in the field and visually classified in the laboratory. The retrieved core was analyzed to identify the type of rock, color, mineral content, bedding planes and other geological and mechanical features of interest in this project. The Rock Quality Designation (RQD) for each rock core run was calculated according to the following equation:

$$RQD = \frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

The RQD value aids in estimating the general quality of the rock and is used in conjunction with other parameters to designate the quality of the rock mass.

Upon completion of drilling, the borings were backfilled with bentonite chips and soil cuttings. Where borings penetrated the existing pavement, an equivalent thickness of cold patch asphalt was used to repair the pavement surface.

During drilling, field personnel prepared field logs showing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved and sealed in glass jars and delivered to the soil laboratory. In the laboratory, the recovered soil and rock samples were visually classified, and select samples from the borings performed for the subject structures were tested, as noted in Table 2.

Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	16
Plastic and Liquid Limits	AASHTO T89, T90	4
Gradation – Sieve/Hydrometer	AASHTO T88	4

The tests performed are necessary to classify existing soil according to the ODOT classification system and to estimate engineering properties of importance in determining foundation design and construction recommendations. Results of the laboratory testing are presented on the individual boring logs in Appendix III. A description of the soil and rock terms used throughout this report is presented in Appendix II.

3.1 Historical Borings

In addition to the borings performed for this project, Rii utilized historical borings in the area of the proposed structures available through the ODOT Transportation Information Mapping System (TIMS). Boring B-011-0-67 was reportedly performed in the vicinity of the proposed Wall 1. The boring was drilled to a depth of approximately 25 feet. The subsurface profile and material encountered in this boring is described in section 4.2, and a copy of the boring log is presented in Appendix IV.

4.0 FINDINGS

Interpreted engineering logs have been prepared based on the field logs, visual examination of samples and laboratory test results. Classification follows the respective version of the ODOT Specifications for Geotechnical Explorations (SGE) at the time the exploration borings were performed. The following is a summary of what was found in the test borings and what is represented on the boring logs.

4.1 Surface Materials

The borings were generally performed in the vicinity of the proposed retaining walls. Boring B-001-0-19 encountered 12 inches of asphalt overlying 4 inches of aggregate base material at the existing ground surface. Boring B-001-8-21 encountered 6.0 inches of topsoil at the existing ground surface.

4.2 Subsurface Soils

Boring B-001-0-19 was drilled within the existing embankment for the I-70 eastbound ramp to I-270 northbound and encountered embankment fill material to a depth of approximately 23 feet below the existing ground surface. The embankment fill material consisted of stiff to very stiff silty clay (ODOT A-6b).

Underlying the existing fill material in boring B-001-0-19 and the surface material in boring B-001-8-21, the natural soils encountered consisted of both cohesive and granular deposits. The natural cohesive soils were described as silt and clay (ODOT A-6a). The natural granular soils were described as gravel with sand, silt and clay (ODOT A-2-6). It should also be noted that rock fragments were encountered in boring B-001-0-19 at depths ranging between approximately 21 feet and 30 feet below the existing ground surface.

Historic boring B-011-0-67 encountered cohesive and granular material identified as clayey sandy gravel (A-6b) and brown and gray sandy gravel and gravelly sand (ODOT A-1-b, A-2-4). It should be noted that the ground conditions reported in the historic boring vary from those encountered in project boring B-001-0-19. Based on the available information, boring B-011-0-67 is reportedly approximately 90 feet southwest of boring B-001-0-19 and at approximately elevation 796 feet.

The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soils encountered ranged from stiff ($1.0 < HP \leq 2.0$ tsf) to hard ($HP > 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 2.0 to over 4.5 tsf (limit of instrument). The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soils encountered were considered medium dense ($10 < N_{60} \leq 30$ blows per foot [bpf]). Blow counts recorded from the SPT sampling within the granular soil deposits ranged from 13 to 24 bpf.

Natural moisture contents of the soil samples tested ranged from 7 to 26 percent. The natural moisture contents of the cohesive soil samples tested for plasticity ranged from 9 percent below to 9 percent above their corresponding plastic limits. In general, the soil exhibited natural moisture contents considered to be significantly below to significantly above optimum moisture levels.

4.3 Bedrock

Bedrock was encountered in borings B-001-0-19 and B-001-8-21 at depths of 33.5 feet and 4.5 feet beneath the existing ground surface, or approximately elevation 769.9 feet and 775.4 feet msl. The bedrock was described as slightly to highly weathered black shale. Additionally, historic boring B-011-0-67 reported top of weathered rock (unclassified) was encountered at the completion depth of approximately 25 feet, or approximately elevation 770.4 feet msl.

4.4 Groundwater

Groundwater was initially encountered during drilling in borings B-001-0-19 at a depth of 36.0 feet below the existing ground surface. Upon completion of drilling, measurable groundwater was observed in boring B-001-0-19 at a depth of 31.6 feet. Groundwater was not encountered in boring B-001-8-21 prior to the introduction of water for rock coring. Groundwater was not reported on the historic log for boring B-011-0-67.

Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

A more comprehensive description of what was encountered during the drilling process may be found in the boring logs in Appendix III.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the drilling and testing program have been used to determine the shear strength parameters for the soil encountered at the site. These parameters have been used to provide recommendations for the design of the retaining wall structures, as well as the construction specifications related to the retaining wall systems and general earthwork recommendations, which are discussed in the following paragraphs.

Design details of the proposed retaining walls were provided the Rii design team. It is understood that the proposed retaining wall for Ramp F (Wall 1) will be a soil nail wall type that will be constructed along the east side of the ramp to accommodate the proposed widening. The proposed wall will be constructed in front of the existing forward abutment of FRA-70-2293, and will require the existing spill through slope to be cut back. Based on proposed plan and profile information provided, the wall height will vary from approximately 5.0 feet to a maximum height of 15.0 feet. The limits of the proposed retaining wall structure extend along the baseline of Ramp F from station 1533+25 to station 1535+70, for a total length of approximately 245 feet.

5.1 Soil Nail Wall Recommendations

The soil nail wall should be designed and constructed in compliance with the specifications outlined in Section 11.12 of the 2020 AASHTO LRFD BDS and FHWA Geotechnical Engineering Circular No. 7 (GEC 7) Soil Nail Walls (FHWA Publication No. FHWA-NHI-14-007).

It should be noted that per ODOT SGE, borings for soil nail walls should be performed both at the location of the wall as well as in the anchor zone. Boring B-001-1-19 was performed in the anchor zone, but no borings were performed along the proposed wall alignment. Therefore, considerations should be given to performing additional borings along the wall alignment and in the anchor zone in order to evaluate soil conditions along the entirety of the wall alignment.

5.1.1 Internal (Snail) Stability Analysis

The soil nail wall was analyzed for internal stability using the Snail software analysis program developed by the California Department of Transportation. The shear strength parameters and nominal bond stresses provided in Table 3 were utilized in the Snail stability analysis for the soil nail wall.

Table 3. Shear Strength Parameters Utilized in Stability Analyses

Material Type	γ (pcf)	$\varphi'^{(1)}$ (°)	$c'^{(1)}$ (psf)	$S_u^{(2)}$ (psf)	$q_n^{(3)}$ (psi)
Stiff to Very Stiff Silty Clay (ODOT A-6b)	125	24	100	1,200 to 3,000	8.0
Very Stiff to Hard Silt and Clay (ODOT A-6a)	130	26	150	3,000 to 4,000	10.0
Shale Bedrock	135	32	4,000	---	18.0

1. *Strength parameters are based on Section 10.6.4.2 of the 2020 AASHTO LRFD BDS for cohesive soils and engineering judgment.*
2. *Undrained shear strength based on SPT and HP values, Section 10.6.4.2 of the 2020 AASHTO LRFD BDS and engineering judgment.*
3. *Per Table C11.9.4.2-2 and C11.9.4.2-3 from Section 11.12 of the 2020 AASHTO LRFD BDS.*

The shear strength parameters for the natural soils were assigned using correlations provided in Section 10.6.4.2 of the 2020 AASHTO LRFD BDS, and based on past experience in the vicinity of the site with projects performed in similar subsurface profiles. A drained cohesion of 100 to 150 psf was considered for the upper cohesive soil layers to limit the length and diameter of the nails required. Based on the soil conditions encountered in the borings, the consideration of this magnitude of drained cohesion is reasonable based on testing results on undisturbed soil samples performed for adjacent projects.

Based on plan information provided, a vertical spacing of 3.0 feet and horizontal spacing of 5.0 feet was utilized in the analysis, and an inclination of 15° from horizontal was considered. The nominal bond strength for each soil and rock layer was determined from Section 11.12 of the 2020 AASHTO LRFD BDS. Based on the conditions encountered in the borings, the soils through which the nails will be installed will consist of stiff to hard cohesive soils (ODOT A-6a and A-6b). An 8.0-inch diameter augered hole with a No. 8 steel reinforcement bar (1.0 in^2) with 60 ksi yield strength was considered for the soil nail cross section.

The internal stability was evaluated for a critical failure surface that intersected the toe of the wall, as well as for the critical failure surface that passed below the toe of the wall. A nail length of 30 feet was analyzed in order to meet stability requirements.

5.1.2 Pullout Capacity

The pullout capacity of a soil nail installed in a grouted nail hole is affected by the size of the nail (i.e., perimeter and length) and the ultimate bond strength, q_u. The bond strength is the mobilized shear resistance along the soil-grout interface. The bond strength is rarely measured in the laboratory and there is no standard laboratory testing procedure that can be used to evaluate bond strength. Therefore, designs are typically based on conservative estimates of the bond strength obtained from field correlation studies and local experience in similar conditions. Tables C11.12.5.2-1 through C11.12.5.2-3 from Section 11.12 of the 2020 AASHTO LRFD BDS, which reference Tables 4.4a, 4.4b, and 4.5 of GEC 7, provide typical values of the ultimate bond strength for drilled and grouted nails installed in various soils and bedrock and using different drilling methods. As a result of variability in the bond strength and dependency on the installation technique, the contract specifications should include a requirement that some percentage of the soil nails be load tested in the field to verify bond strength design.

Based on the conditions encountered in the borings, the soils through which the nails will be installed will consist of cohesive soil comprised of stiff to hard cohesive soils (ODOT A-6a and A-6b). Utilizing Table C11.12.5.2-2 from Section 11.12 of the 2020 AASHTO LRFD BDS for fine-grained soils comprised of stiff clay, a nominal bond strength of 8.0 to 10.0 psi (1,152 to 1,440 psf) for the cohesive soils was utilized for design of the soil nails. Considering a drilled diameter of 8.0 inches, a nominal pullout capacity of 2.41 and 3.02 kips per lineal foot (klf) was utilized in the analysis.

5.1.3 Sliding Stability

Sliding stability analysis considers the ability of the soil nail wall to resist sliding along the base of the retained system in response to lateral earth pressures behind the soil nails. Sliding failure may occur when additional lateral earth pressures, mobilized by the excavation, exceed the sliding resistance along the base. The sliding resistance consists of two shear strength components, cohesion and internal friction angle as defined in Section 10.6.3.4 of the 2020 AASHTO LRFD BDS. For long-term stability, the effective friction angle, φ', provided in Table 3 for the appropriate soil type, should be used for design. For short-term stability, the undrained shear strength, S_u, provided in Table 3 should be used for design where cohesive soils are present along the base of the wall, and the effective friction angle should be used where granular soils are present. Recommended earth pressure coefficients for the soils encountered at the site are presented in Table 6 and Table 7 in Section 5.2.

Due to the required length of the soil nails for internal stability, the overall mass considered for sliding stability was determined by projecting the proposed 2:1 backslope behind the wall up to intersect with the roadway. For this scenario, it is considered that the soil nails will not provide any contribution to the stability of the sliding mass. Based on the soil mass considered and utilizing the soil parameters listed in Section 5.1.1 for the retained embankment material, the resultant horizontal forces on the back of the soil nail wall **will not exceed** the factored shear resistance at the strength limit state under drained and undrained conditions.

5.1.4 Bearing Capacity (Basal Heave)

Bearing capacity analyses are routinely not necessary for cases where soft soils (e.g., $S_u \leq 500$ psf) are not present at the bottom of the excavation. An exception to this general rule-of-thumb is when large loads are imposed behind the proposed soil nail wall. For this case, since the retaining wall will be supporting the soil beneath a bridge foundation element, a bearing capacity analysis is recommended regardless of the soil conditions. The bearing capacity should be evaluated using the methodology outlined in Section 5.6.6 of GEC 7 using the undrained shear strength parameters provided in Table 3.

Due to the required length of the soil nails for internal stability, the overall mass considered for bearing stability was determined by projecting the proposed 2:1 backslope behind the wall up to intersect with the roadway. Rii performed a verification of the bearing pressure exerted on the subgrade material for the height and width of soil mass considered. Based on the soil mass considered, the factored equivalent bearing pressure exerted below the wall **will not exceed** the factored bearing resistance at the strength limit state under drained or undrained conditions.

5.1.5 Global Stability

A slope stability analysis was performed to check the global stability of the wall along the alignment. As per the 2020 AASHTO LRFD BDS, safety against soil failure shall be evaluated at the service limit state. Soil parameters utilized in global stability analysis are presented in Section 5.1.1. The computer software program Slide manufactured by Rocscience Inc. was utilized to perform the analysis.

Per Section 11.6.3.7 of the 2020 AASHTO LRFD BDS, overall (global) stability for retaining walls that are integrated with or supporting structural foundations or elements is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor φ=0.65 is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.5 is obtained. Based on the soil nail configuration, the resulting factor of safety under drained conditions (long-term stability) along the alignment was greater than 1.5.

5.1.6 Corrosivity

Corrosivity testing was performed on samples retrieved from boring B-001-0-19 on samples between the depths of 1.0 feet to 20.0 feet below existing grade. The pH of the soils ranged was 7.49. The sulfate concentration 840 parts per million (ppm). The soluble chloride ion content in the soil was 420 mg/kg. The resistivity of the soil 790 ohm-cm. Based on the results of the resistivity, the correlation with ferrous metal and corrosivity category is developed.

Table 4. Correlation between Electrical Resistivity and Corrosivity

Soil Resistivity (ohm – cm)	Corrosivity Category
Greater than 10,000	Mildly corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

1. *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166-167*

Based on soil resistivity testing, the soils for this project are considered severely corrosive.

Table 5. Soil Resistivity and Corrosivity Category

Borings / Sample	Soil Resistivity (ohm-cm)	Corrosivity Category
B-001-0-19 SS-1 through SS-10	790	Severely Corrosive

In general, other factors, such as pH, sulfate and moisture content of the soil also affect the corrosion rate of the soil nails. Results of the soil resistivity, sulfate content, and chloride concentration indicate that these levels in the soil are severely corrosive. Based on the results of the corrosivity testing performed, Rii recommends Class A bar encapsulation corrosion protection in accordance with Section 11.12.8 of the 2020 AASHTO LRFD BDS.

5.1.7 Final Evaluation

Based on the results of the internal, external and global stability analysis performed for the soil nail wall, the recommended minimum nail lengths are presented in Section 5.1.1. Internal stability of the soil nail wall with a failure surface extending below the bottom of the wall was the controlling factor in the determination of the recommended nail lengths. Additionally, the controlling component in the internal stability analysis was pullout of the nails. However, it should be noted that yielding of the bar steel may control if a lower grade of steel than 60 ksi or smaller bar diameter is used in the final design.

Calculations for internal (Snail), external (bearing and sliding resistance) and global (Slide) stability of the soil nail wall are provided in Appendix V.

5.2 Lateral Earth Pressure Parameters

For the soil types encountered in the borings, the “in-situ” unit weight (γ), cohesion (c), effective angle of friction (φ’), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 6 and Table 7.

Table 6. Estimated Undrained Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ	k_a	k_o	k_p
Stiff Cohesive Soil	120	2,000	0°	N/A	N/A	N/A
Very Stiff to Hard Cohesive Soil	125	3,000	0°	N/A	N/A	N/A
Loose to Medium Dense Granular Soil	120	0	28°	0.32	0.53	5.07
Dense to Very Dense Granular Soil	130	0	34°	0.25	0.44	8.00
Compacted Cohesive Engineered Fill	120	2,000	0°	N/A	N/A	N/A
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

1. *When below groundwater table, use effective unit weight, $\gamma' = \gamma$ - 62.4 pcf and add hydrostatic water pressure.*

Table 7. Estimated Drained Soil Parameters for Design

Soil Type	γ (pcf) [1]	c (psf)	φ'	k_a	k_o	k_p
Stiff Natural Cohesive Soil	112	0	25°	0.36	0.58	4.26
Very Stiff to Hard Natural Cohesive Soil	125	0	27°	0.33	0.55	4.80
Loose to Medium Dense Granular Soil	120	0	28°	0.32	0.53	5.07
Dense to Very Dense Granular Soil	130	0	34°	0.25	0.44	8.00
Compacted Cohesive Engineered Fill	120	0	30°	0.30	0.50	5.58
Compacted Granular Engineered Fill	120	0	32°	0.27	0.47	6.82

1. *When below groundwater table, use effective unit weight, $\gamma' = \gamma$ - 62.4 pcf and add hydrostatic water pressure.*

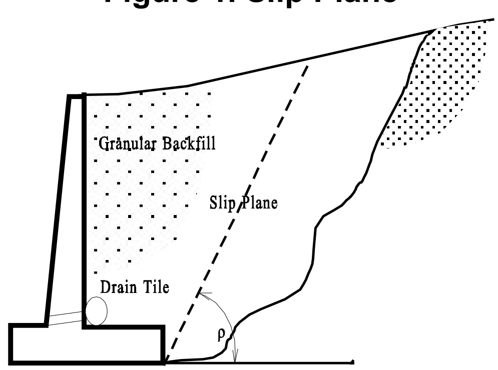
These parameters are considered appropriate for the design of subsurface walls and excavation support systems. Subsurface structures (where the top of the structure is restrained from movement) should be designed based on at-rest conditions. For proposed wing walls or temporary retaining structures (where the top of the structure is allowed to move), earth pressure distributions should be based on active conditions (k_a) and passive pressure (k_p). The values in this table have been estimated from correlation charts based on minimum standards specified for compacted engineered fill materials. These recommendations do not take into consideration the effect of any surcharge loading or a sloped ground surface (a flat surface is assumed). Earth pressures on excavation support systems will be dependent on the type of sheeting and method of bracing or anchorage.

In order to alleviate the build-up of hydrostatic pressure behind the walls, a minimum of 2.0 feet of clean free-draining granular fill (i.e., No. 57 gravel) should be placed full depth behind the walls. If granular fill other than No. 57 gravel is used, it should not have more than 8 percent (by weight) passing the No. 200 screen, and should be compacted to 95 percent of the maximum dry density as determined by the Standard Proctor Test (ASTM D698). A perforated, corrugated drain tile, wrapped with filter fabric, should be placed along the perimeter at the base of the walls for drainage purposes. A clay cap (minimum 1.0-foot thick) should be placed overtop the granular backfill to deter inflow of the surface water. The drainage system should properly outlet to a sewer or to a properly sized sump pump system.

Temporary retaining structures should be designed using the undrained soil parameters provided in Table 6, and the design should follow all applicable guidelines for the type of retaining structure utilized. Permanent retaining structures should be designed using the drained soil parameters provided in Table 7. Regardless of whether the retaining structure is temporary or permanent, the effective unit weight ($\gamma' = \gamma$ - 62.4 pcf) plus the hydrostatic water pressure ($\gamma_w * h_w$, where h_w is the height of water behind the wall above the base of the wall) should be utilized below the design groundwater level. The lateral earth pressure coefficients should only be applied to the horizontal pressure resulting from the effective overburden pressure, and should not be applied to the hydrostatic water pressure.

The 2.0 feet of free draining material placed behind the wall prevents the formation of hydrostatic pressures as noted above. However, unless the free draining granular backfill is placed beyond the slip plane (see Figure 1), it has no influence on the equivalent fluid weight of the soil. If free-draining granular fill (meeting the requirements listed above) is to be placed beyond the slip plane (ρ=45° for at-rest conditions; ρ=45°+φ/2 for active conditions), the values presented for the compacted granular engineered fill can be employed, consequently lowering the pressures on the wall.

Figure 1. Slip Plane



Backfill Rankine Zone with Select Backfill

5.3 Construction Considerations

All site work shall conform to local codes, and to the latest ODOT CMS, including that all excavation and embankment preparation and construction should follow ODOT Item 200 (Earthwork).

5.3.1 Excavation Considerations

All excavations should be shored / braced or laid back at a safe angle in accordance to Occupational Safety and Health Administration (OSHA) guidelines. During excavation, if slopes cannot be laid back to OSHA Standards due to adjacent structures or other obstructions, temporary shoring may be required. The following table should be utilized as a general guide for implementing OSHA guidelines when estimating excavation back slopes at the various boring locations. Actual excavation back slopes must be field verified by qualified personnel at the time of excavation in strict accordance with OSHA guidelines.

Table 8. Excavation Back Slopes

Soil	Maximum Back Slope	Notes
Soft to Medium Stiff Cohesive	1.5 : 1.0	Above Ground Water Table and No Seepage
Stiff Cohesive	1.0 : 1.0	Above Ground Water Table and No Seepage
Very Stiff to Hard Cohesive	0.75 : 1.0	Above Ground Water Table and No Seepage
All Granular & Cohesive Soil Below Ground Water Table or with Seepage	1.5 : 1.0	None
Rock to 3.0' +/- below Auger Refusal	0.75 : 1.0	Above Ground Water Table and No Seepage
Stable Rock	Vertical	Above Ground Water Table and No Seepage

5.3.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater seepage is not anticipated during construction. Where groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

The above recommendations are predicated upon construction inspection by a qualified soil technician under the direct supervision of a professional geotechnical engineer. Adequate testing and inspection during construction are considered necessary to assure an adequate foundation system and are part of our recommendations.

The recommendations for this project were developed utilizing soil and bedrock information obtained from the test borings that were made at the proposed site. At this time we would like to point out that soil borings only depict the soil and bedrock conditions at the specific locations and time at which they were made. The conditions at other locations on the site may differ from those occurring at the boring locations.

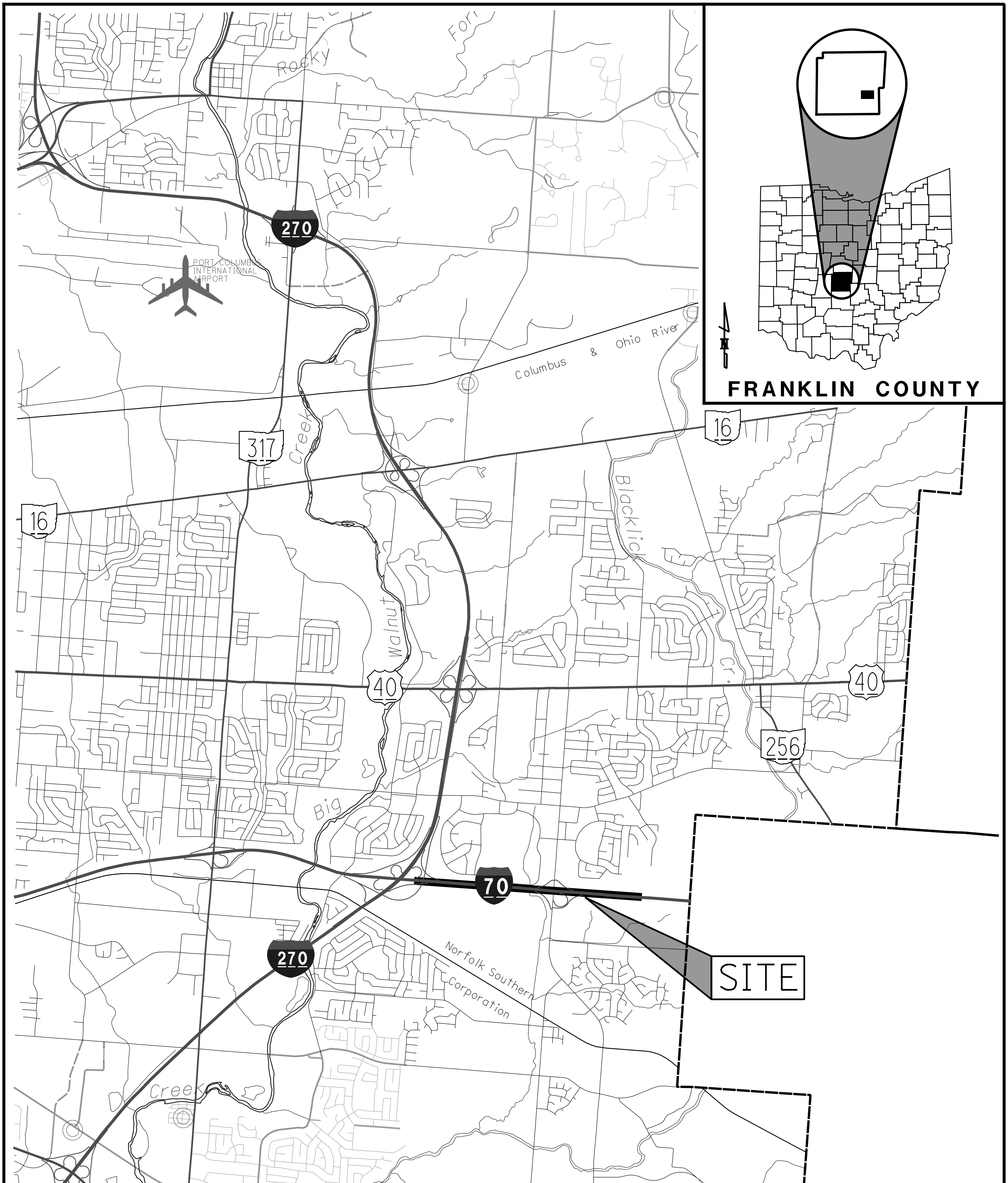
The conclusions and recommendations herein have been based upon the available soil information and the preliminary design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be brought to the attention of the geotechnical engineer to determine whether any changes in the foundation or earthwork recommendations are necessary. If deviations from the noted subsurface conditions are encountered during construction, they should also be brought to the attention of the geotechnical engineer.

The scope of our services does not include any environmental assessment or investigation for the presence or absence or hazardous or toxic materials in the soil, groundwater or surface water within or beyond the site studied. Any statements in this report or on the test boring logs regarding odors, staining of soils or other unusual conditions observed are strictly for the information of our client.

Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. Resource International is not responsible for the conclusions, opinions or recommendations made by others based upon the data included.

APPENDIX I

VICINITY MAP AND BORING PLAN

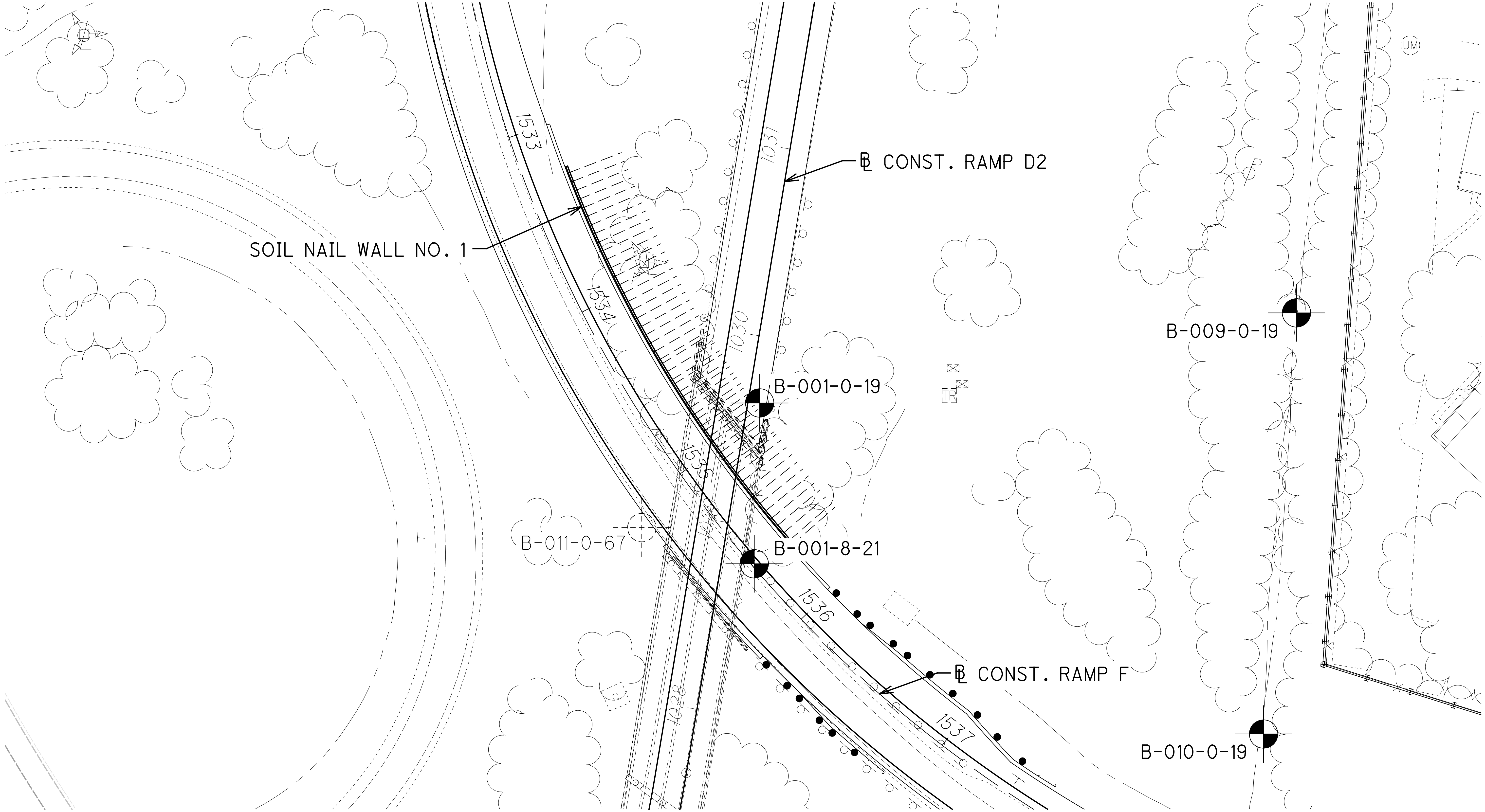


VICINITY MAP

FRA-70-22.85

COLUMBUS, OHIO

RII PROJECT NO. W-17-140	DRAWN JAS	
	REVIEWED PPM	
SCALE: 1"=5000' (0 — 2500 — 5000)	DATE 1/15/2021	



BORING PLAN

SOIL NAIL WALL NO. 1

FRANKLIN COUNTY, OHIO

RII PROJECT NO. W-17-140

SCALE: 1"=50'

0 25 50

N

DRAWN RRM

REVIEWED BRT

DATE 4/02/22

RESOURCE INTERNATIONAL

APPENDIX II

DESCRIPTION OF SOIL AND ROCK TERMS

DESCRIPTION OF SOIL TERMS

The following terminology was used to describe soils throughout this report and is generally adapted from ASTM 2487/2488 and ODOT Specifications for Geotechnical Explorations.

Granular Soils - The relative compactness of granular soils is described as:
ODOT A-1, A-2, A-3, A-4 (non-plastic) or USCS GW, GP, GM, GC, SW, SP, SM, SC, ML (non-plastic)

Description	Blows per foot – SPT (N_{60})		
Very Loose	Below		5
Loose	5	-	10
Medium Dense	11	-	30
Dense	31	-	50
Very Dense	Over		50

Cohesive Soils - The relative consistency of cohesive soils is described as:
ODOT A-4, A-5, A-6, A-7, A-8 or USCS ML, CL, OL, MH, CH, OH, PT

Description	Unconfined Compression (tsf)		
Very Soft	Less than		0.25
Soft	0.25	-	0.5
Medium Stiff	0.5	-	1.0
Stiff	1.0	-	2.0
Very Stiff	2.0	-	4.0
Hard	Over		4.0

Gradation - The following size-related denominations are used to describe soils:

Soil Fraction		USCS Size	ODOT Size
Boulders		Larger than 12"	Larger than 12"
Cobbles		12" to 3"	12" to 3"
Gravel	coarse	3" to ¾"	3" to ¾"
	fine	¾" to 4.75 mm (¾" to #4 Sieve)	¾" to 2.0 mm (¾" to #10 Sieve)
Sand	coarse	4.75 mm to 2.0 mm (#4 to #10 Sieve)	2.0 mm to 0.42 mm (#10 to #40 Sieve)
	medium	2.0 mm to 0.42 mm (#10 to #40 Sieve)	-
	fine	0.42 mm to 0.074 mm (#40 to #200 Sieve)	0.42 mm to 0.074 mm (#40 to #200 Sieve)
Silt		0.074 mm to 0.005 mm (#200 to 0.005 mm)	0.074 mm to 0.005 mm (#200 to 0.005 mm)
Clay		Smaller than 0.005 mm	Smaller than 0.005 mm

Modifiers of Components - Modifiers of components are as follows:

Term	Range		
Trace	0%	-	10%
Little	10%	-	20%
Some	20%	-	35%
And	35%	-	50%

Moisture Table - The following moisture-related denominations are used to describe cohesive soils:

Term	Range - USCS	Range - ODOT
Dry	0% to 10%	Well below Plastic Limit
Damp	>2% below Plastic Limit	Below Plastic Limit
Moist	2% below to 2% above Plastic Limit	Above PL to 3% below LL
Very Moist	>2% above Plastic Limit	
Wet	≥ Liquid Limit	3% below LL to above LL

Organic Content – The following terms are used to describe organic soils:

Term	Organic Content (%)
Slightly organic	2-4
Moderately organic	4-10
Highly organic	>10

Bedrock – The following terms are used to describe the relative strength of bedrock:

Description	Field Parameter
Very Weak	Can be carved with knife and scratched by fingernail. Pieces 1 in. thick can be broken by finger pressure.
Weak	Can be grooved or gouged with knife readily. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Can be grooved or gouged 0.05 in deep with knife. 1 in. size pieces from hard blows of geologist hammer.
Moderately Strong	Can be scratched with knife or pick. 1/4 in. size grooves or gouges from blows of geologist hammer.
Strong	Can be scratched with knife or pick with difficulty. Hard hammer blows to detach hand specimen.
Very Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to detach hand specimen.
Extremely Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to chip hand specimen.

DESCRIPTION OF ROCK TERMS

The following terminology was used to describe the rock throughout this report and is generally adapted from ASTM D5878 and the ODOT Specifications for Geotechnical Explorations.

Weathering – Describes the degree of weathering of the rock mass:

Description	Field Parameter
Unweathered	No evidence of any chemical or mechanical alteration of the rock mass. Mineral crystals have a right appearance with no discoloration. Fractures show little or not staining on surfaces.
Slightly Weathered	Slight discoloration of the rock surface with minor alterations along discontinuities. Less than 10% of the rock volume presents alteration.
Moderately Weathered	Portions of the rock mass are discolored as evident by a dull appearance. Surfaces may have a pitted appearance with weathering "halos" evident. Isolated zones of varying rock strengths due to alteration may be present. 10 to 15% of the rock volume presents alterations.
Highly Weathered	Entire rock mass appears discolored and dull. Some pockets of slightly to moderately weathered rock may be present and some areas of severely weathered materials may be present.
Severely Weathered	Majority of the rock mass reduced to a soil-like state with relic rock structure discernable. Zones of more resistant rock may be present but the material can generally be molded and crumbled by hand pressures.

Strength of Bedrock – The following terms are used to describe the relative strength of bedrock:

Description	Field Parameter
Very Weak	Can be carved with knife and scratched by fingernail. Pieces 1 in. thick can be broken by finger pressure.
Weak	Can be grooved or gouged with knife readily. Small, thin pieces can be broken by finger pressure.
Slightly Strong	Can be grooved or gouged 0.05 in deep with knife. 1 in. size pieces from hard blows of geologist hammer.
Moderately Strong	Can be scratched with knife or pick. 1/4 in. size grooves or gouges from blows of geologist hammer.
Strong	Can be scratched with knife or pick with difficulty. Hard hammer blows to detach hand specimen.
Very Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to detach hand specimen.
Extremely Strong	Cannot be scratched by knife or pick. Hard repeated blows of geologist hammer to chip hand specimen.

Bedding Thickness – Description of bedding thickness as the average perpendicular distances between bedding surfaces:

Description	Thickness
Very Thick	Greater than 36 inches
Thick	18 to 36 inches
Medium	10 to 18 inches
Thin	2 to 10 inches
Very Thin	0.4 to 2 inches
Laminated	0.1 to 0.4 inches
Thinly Laminated	Less than 0.1 inches

Fracturing – Describes the degree and condition of fracturing (fault, joint, or shear):

Degree of Fracturing

Description	Spacing
Unfractured	Greater than 10 feet
Intact	3 to 10 feet
Slightly Fractured	1 to 3 feet
Moderately Fractured	

Aperture Width

Description	Width
Open	Greater than 0.2 inches
Narrow	0.05 to 0.2 inches
Tight	Less than 0.05 inches

Surface Roughness

Description	Criteria
Very Rough	Near vertical steps and ridges occur on surface
Slightly Rough	Asperities on the surfaces distinguishable
Slickensided	Surface has smooth, glassy finish, evidence of Striations

RQD – Rock Quality Designation (calculation shown in report) and Rock Quality (ODOT, GB 3, January 13, 2006):

RQD %	Rock Index Property Classification (based on RQD, not slake durability index)
0 – 25%	Very Poor
26 – 50%	Poor
51 – 70%	Fair
71 – 85%	Good
86 – 100%	Very Good

CLASSIFICATION OF SOILS

Ohio Department of Transportation

(The classification of a soil is found by proceeding from top to bottom of the chart. The first classification that the test data fits is the correct classification.)

SYMBOL	DESCRIPTION	Classifcation AASHTO	Classifcation OHIO	LL_O/LL × 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
F S	Fine Sand	A-3			51 Min.	10 Max.	NON-PLASTIC	NON-PLASTIC	0	
	Coarse and Fine Sand	--	A-3a			35 Max.		6 Max.	0	Min. of 50% combined coarse and fine sand sizes
	Gravel and/or Stone Fragments with Sand and Silt	A-2-4				35 Max.	40 Max.	10 Max.	0	
		A-2-5					41 Min.			
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6				35 Max.	40 Max.	11 Min.	4	
		A-2-7					41 Min.			
	Sandy Silt	A-4	A-4a	76 Min.		36 Min.	40 Max.	10 Max.	8	Less than 50% silt sizes
	Silt	A-4	A-4b	76 Min.		50 Min.	40 Max.	10 Max.	8	50% or more silt sizes
	Elastic Silt and Clay	A-5		76 Min.		36 Min.	41 Min.	10 Max.	12	
	Silt and Clay	A-6	A-6a	76 Min.		36 Min.	40 Max.	11 - 15	10	
	Silty Clay	A-6	A-6b	76 Min.		36 Min.	40 Max.	16 Min.	16	
	Elastic Clay	A-7-5		76 Min.		36 Min.	41 Min.	≦LL-30	20	
	Clay	A-7-6		76 Min.		36 Min.	41 Min.	>LL-30	20	
	Organic Silt	A-8	A-8a	75 Max.		36 Min.				W/o organics would classify as A-4a or A-4b
	Organic Clay	A-8	A-8b	75 Max.		36 Min.				W/o organics would classify as A-5, A-6a, A-6b, A-7-5 or A-7-6

MATERIAL CLASSIFIED BY VISUAL INSPECTION

- Sod and Topsoil
- Pavement or Base
- Uncontrolled Fill (Describe)
- Bouldery Zone
- P Peat

* Only perform the oven-dried liquid limit test and this calculation if organic material is present in the sample.

APPENDIX III

PROJECT BORING LOGS

BORING LOGS

Definitions of Abbreviations

AS	=	Auger sample
GI	=	Group index as determined from the Ohio Department of Transportation classification system
HP	=	Unconfined compressive strength as determined by a hand penetrometer (tons per square foot)
LL_o	=	Oven-dried liquid limit as determined by ASTM D4318. Per ASTM D2487, if LL_o/LL is less than 75 percent, soil is classified as “organic”.
LOI	=	Percent organic content (by weight) as determined by ASTM D2974 (loss on ignition test)
PID	=	Photo-ionization detector reading (parts per million)
QR	=	Unconfined compressive strength of intact rock core sample as determined by ASTM D2938 (pounds per square inch)
QU	=	Unconfined compressive strength of soil sample as determined by ASTM D2166 (pounds per square foot)
RC	=	Rock core sample
REC	=	Ratio of total length of recovered soil or rock to the total sample length, expressed as a percentage
RQD	=	Rock quality designation – estimate of the degree of jointing or fracture in a rock mass, expressed as a percentage:

$$\frac{\sum \text{segments equal to or longer than 4.0 inches}}{\text{core run length}} \times 100$$

S	=	Sulfate content (parts per million)
SPT	=	Standard penetration test blow counts, per ASTM D1586. Driving resistance recorded in terms of blows per 6-inch interval while letting a 140-pound hammer free fall 30 inches to drive a 2-inch outer diameter (O.D.) split spoon sampler a total of 18 inches. The second and third intervals are added to obtain the number of blows per foot (N_m).
N_{60}	=	Measured blow counts corrected to an equivalent (60 percent) energy ratio (ER) by the following equation: $N_{60} = N_m*(ER/60)$
SS	=	Split spoon sample
2S	=	For instances of no recovery from standard SS interval, a 2.5 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 2S sampling are not correlated with N_{60} values.
3S	=	Same as 2S, but using a 3.0 inch O.D. split spoon sampler.
TR	=	Top of rock
W	=	Initial water level measured during drilling
▼	=	Water level measured at completion of drilling

Classification Test Data

Gradation (as defined on Description of Soil Terms):

GR	=	% Gravel
SA	=	% Sand
SI	=	% Silt
CL	=	% Clay

Atterberg Limits:

LL	=	Liquid limit
PL	=	Plastic limit
PI	=	Plasticity Index

WC	=	Water content (%)

RESOURCE INTERNATIONAL, INC.

PROJECT: FRA-070-22.85	DRILLING FIRM / OPERATOR: RII / LH	DRILL RIG: CME 55 (386345)	STATION / OFFSET: 1029+65.04 / 6.1' RT	EXPLORATION ID **B-001-0-19**
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: RII / JK	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D2	
PID: 98232 SFN: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/4/18	ELEVATION: 803.4 (MSL) EOB: 38.7 ft.	PAGE 1 OF 2
START: 8/4/20 END: 8/4/20	SAMPLING METHOD: SPT	ENERGY RATIO (%): 90	LAT / LONG: 39.934994, -82.848710	

MATERIAL DESCRIPTION AND NOTES	ELEV. 803.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
1.0' - ASPHALT (12.0")	802.4																	
0.3' - AGGREGATE BASE (4.0")	802.1																	
FILL: STIFF TO VERY STIFF, BROWN, GRAY AND DARK BROWN **SILTY CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST.		1–2	5 7 5	18	50	SS-1	4.00	-	-	-	-	-	-	-	-	18	A-6b (V)	
		4–5	7 3 4	11	44	SS-2	2.00	-	-	-	-	-	-	-	-	24	A-6b (V)	
		6–7	6 4 5	14	58	SS-3	2.75	-	-	-	-	-	-	-	-	20	A-6b (V)	
		9–10	4 4 6	15	69	SS-4	3.00	7	16	14	30	33	37	19	18	18	A-6b (9)	
		11–12	5 6 7	20	92	SS-5	3.50	-	-	-	-	-	-	-	-	15	A-6b (V)	
		14–15	4 8 10	27	33	SS-6	3.50	-	-	-	-	-	-	-	-	13	A-6b (V)	
		16–17	10 9 10	29	67	SS-7	3.00	-	-	-	-	-	-	-	-	20	A-6b (V)	
		19–20	12 15 14	44	61	SS-8	3.00	1	6	14	38	41	38	19	19	17	A-6b (12)	
-TRACE LIMESTONE FRAGMENTS IN SS-9	780.4	21–22	9 15 24	59	33	SS-9	3.25	-	-	-	-	-	-	-	-	14	A-6b (V)	
VERY STIFF TO HARD, BROWN, GRAY AND DARK BROWN TO BLACK **SILT AND CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST. -LIMESTONE FRAGMENTS IN SS-10		24–25	10 11 13	36	44	SS-10	3.00	2	11	16	37	34	29	16	13	7	A-6a (8)	
-SHALE FRAGMENTS IN SS-11		29	8 13 10	35	44	SS-11	-	-	-	-	-	-	-	-	-	10	A-6a (V)	

PID: 98232	SFN:	PROJECT: FRA-070-22.85	STATION / OFFSET: 102965.04, 6' RT.	START: 8/4/20	END: 8/4/20	PG 2 OF 2	B-001-0-19

MATERIAL DESCRIPTION AND NOTES	ELEV. 773.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
VERY STIFF TO HARD, BROWN, GRAY AND DARK BROWN TO BLACK **SILT AND CLAY**, SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST. *(continued)*	769.9	▽ 771.8 / 31 / 32 / 33 / TR																
SHALE : BLACK, HIGHLY WEATHERED.		34 / 35	10 17 23	60	69	SS-12	-	-	-	-	-	-	-	-	-	8	Rock (V)	
		W 767.4 / 36 / 37 / 38																
	764.6	EOB	50/3"	-	100	SS-13	-	-	-	-	-	-	-	-	-	13	Rock (V)	

NOTES: GROUNDWATER ENCOUNTERED INITIALLY @ 36.0' AND AT COMPLETION @ 31.6'; CAVE-IN DEPTH @ 34.8'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH 50 LBS. BENTONITE CHIPS AND SOIL CUTTINGS. .

RESOURCE INTERNATIONAL, INC.

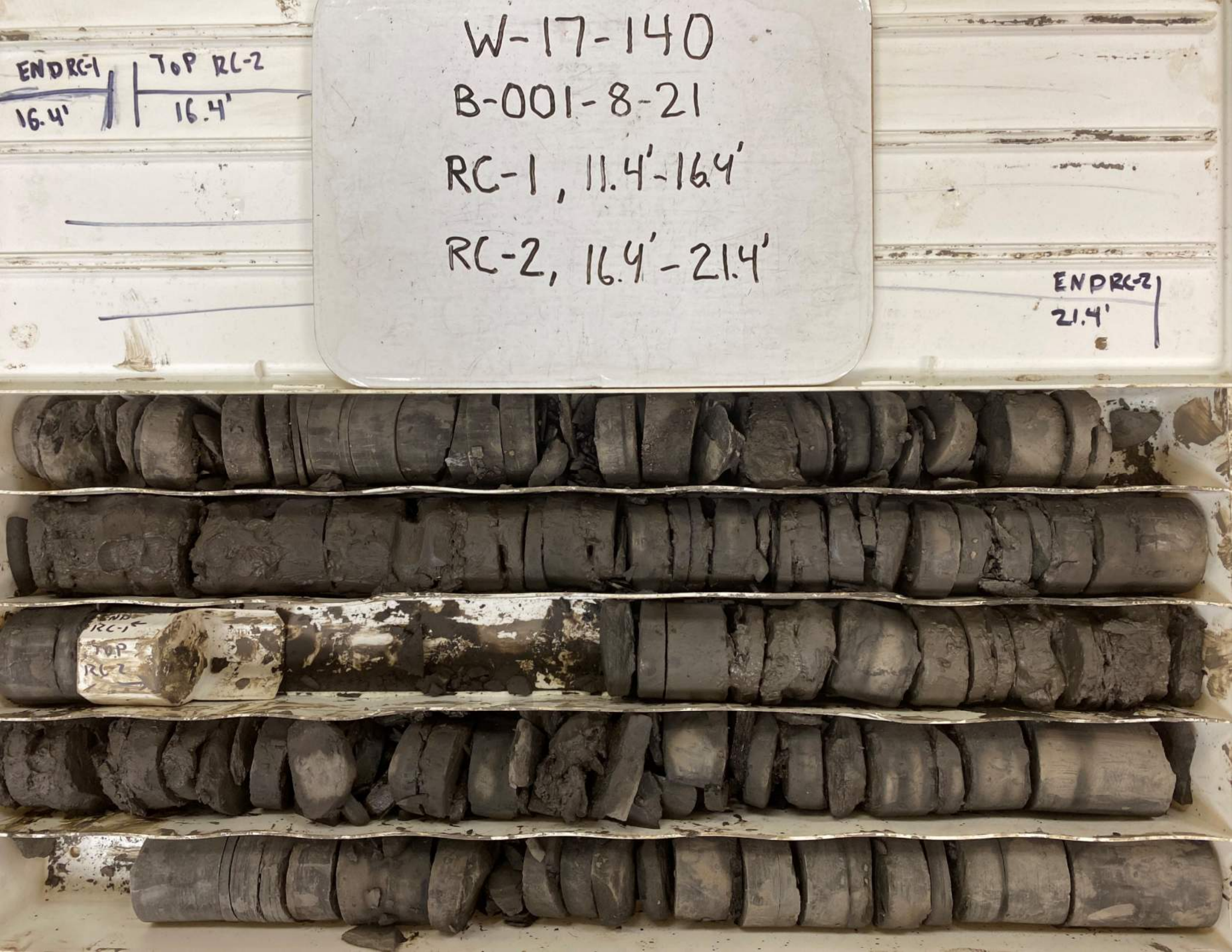
PROJECT: FRA-070-22.85	DRILLING FIRM / OPERATOR: RII / TG	DRILL RIG: MOBILE B53 (SN 386345)	STATION / OFFSET: 1028+80.78 / 17.2' RT	EXPLORATION ID B-001-8-21
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: RII / MJ	HAMMER: AUTOMATIC	ALIGNMENT: BL RAMP D2	
PID: 98232 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 9/14/20	ELEVATION: 779.9 (MSL) EOB: 21.4 ft.	PAGE
START: 11/30/21 END: 11/30/21	SAMPLING METHOD: SPT	ENERGY RATIO (%): 83.6	LAT / LONG: 39.934761, -82.848719	1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 779.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
6.0" - TOPSOIL	779.4																	
MEDIUM DENSE, DARK BROWN **GRAVEL WITH SAND, SILT, AND CLAY**, MOIST.			3 4 7	15	89	SS-1	-	-	-	-	-	-	-	-	-	12	A-2-6 (V)	
	775.4	TR	6 7 9	22	75	SS-2A	-	40	13	12	21	14	28	17	11	14	A-2-6 (0)	
SHALE : DARK GRAY, HIGHLY WEATHERED.						SS-2B	-	-	-	-	-	-	-	-	-	13	Rock (V)	
			23 27 23	70	100	SS-3	-	-	-	-	-	-	-	-	-	-	Rock (V)	
			46 50/5"	-	100	SS-4	-	-	-	-	-	-	-	-	-	-	Rock (V)	
	768.5		50/5"	-	100	SS-5	-	-	-	-	-	-	-	-	-	-	Rock (V)	
SHALE : BLACK, SLIGHTLY WEATHERED, SLIGHTLY STRONG, THIN BEDDED, FISSILE, FRACTURED, NARROW, SLIGHTLY ROUGH, BLOCKY/DISTURBED/SEAMY, GOOD. -CLAY SEAMS PRESENT THROUGHOUT			0		84	NQ-6											CORE	
	758.5	EOB	0		96	NQ-7											CORE	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 25 LB. BENTONITE CHIPS AND SOIL CUTTINGS .

00-2021 NEW STA ODOT BORING LOG (8.5X11) - OH DOT.GDT - 4/8/22 15:09 - U:\GI8\PROJECTS\2017\W-17-140.GPJ


W-17-140
B-001-8-21
RC-1, 11.4'-16.4'
RC-2, 16.4'-21.4'
END RC-1
16.4'
TOP RC-2
16.4'
END RC-2
21.4'
END RC-1
TOP RC-2

APPENDIX IV

HISTORIC BORING LOGS

LOG OF BORING

Date Started 6-2-67 | Sampler Type SS | Dia. 1 3/8" | Water Elev. ______

Date Completed 6-2-67 | Casing: Length 20' | Dia. 3 1/2"

Boring No. B-11 | Station & Offset 1028+78, 28' Lt. (5th Pier) | Surface Elev. 796.0'

Elev.	Depth	Std. Pen. (N)	Rec. ft.	Loss ft.	Description	Sample No.	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class
796.0	0														
	2														
	4														
791.0	6	3/4			Brown Clayey Sandy Gravel	1	39	12	12	14	23	40	19	15	A-6h
	8														
786.0	10	4/14			Brown Silty Sandy Gravel	2	49	19	13	-19	-	NP	NP	11	A-1-b
783.5	12 / 14				Brown Sandy Gravel (Wash Sample)	3	50	27	14	- 9	-	NP	NP	15	A-1-a
781.0	16				Brown Sand- (Wash Sample) (Sample Heaved 4' in Casing)	4	8	83	8	- 1	-	NP	NP	28	A-1-b
778.5	18				Brown Gravelly Sand (Wash Sample)	5	19	70	6	-5	-	NP	NP	24	A-1-b
776.0	20	15/19			Gray Silty Sandy Gravel	6	48	17	12	15	8	-	-	14	-
	22														
771.0	24				TOP OF WEATHERED ROCK — BOTTOM OF BORING										

APPENDIX V

CORROSIVITY TEST RESULTS

RESOURCE INTERNATIONAL, INC.
Engineering Consultants

6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

Project Name: FRA-070-22.85
Project No.: W-17-140
Boring: B-001-0-19

Date: 3/31/2022
Tested by: EM

pH of Soils
(ASTM D4972 - Method A)

Sieve # 10: X

Boring ID	Sample ID	Depth (feet)	pH in water	pH in calcium chloride sol.	Temp. (C)
Composite Sample	SS-1 to SS-9	1.0'-20.0'	8.05	7.49	22.2

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Project No.: W-17-140
Boring: B-001-0-19

Date: 3/31/2022
Tested by: EM

Minimum Laboratory Soil Resistivity (AASHTO T-288)

Boring ID	Sample ID	Depth (feet)	Resistivity (ohm/cm)
Composite Sample	SS-1 to SS-9	1.0'-20.0'	790

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Project No.: W-17-140
Boring: B-001-0-19

Date: 3/31/2022
Tested by: EM

Testing for Sulfate Content in Soil (ODOT S 1122)

Boring ID	Sample ID	Depth (feet)	Sulfate content (ppm)
Composite Sample	SS-1 to SS-9	1.0'-20.0'	840

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Project Name: FRA-070-22.85
Project No.: W-17-140
Boring: B-001-0-19

Date: 3/31/2022
Tested by: EM

Measurement of Oxidation-Reduction Potential (ORP) of Soil

(ASTM C1498 1:1 soil in water mixture)

Boring ID	Sample ID	Depth (feet)	ORP (mV)
Composite Sample	SS-1 to SS-9	1.0'-20.0'	216

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Project Name: FRA-070-22.85
Project No.: W-17-140
Boring: B-001-0-19

Date: 3/31/2022
Tested by: EM

Testing for Water-Soluble Chloride Ion in Soil (AASHTO T-291)

Method of Testing: Method A

Boring ID	Sample ID	Depth (feet)	Water-soluble Chloride Ion in Soil (mg/kg)
Composite Sample	SS-1 to SS-9	1.0'-20.0'	420

APPENDIX VI

CALCULATIONS – WALL 1

Snail

Version: 2.2.2

File Information

File Name: FRA-70-22.85 Wall 1 - 4 rows, 15ft dek.snz
Run Date: 12/22/22
Run Time: 10:26:38

Project Information

Description: FRA-70-22.85
Location:
EA:
Project ID: W-17-140
Wall No.: 1
Structure No.:
Station:
Engineer: D. Karch
Designer

Comments:

Soil Nail wall analysis for B-001-0-19

Geometry

Layout:

Reference Point:

At: Toe of Wall
Distance From Origin: 0.00 feet
Elevation Above Origin: 776.60 feet

Wall Dimensions:

Wall Height: 15.00 feet
Facing Angle: 90.00 degrees
Facing Batter: 0.000 :12 H:V

Ground Surface:

Number of lines that define the ground surface above the wall: 3

No.	Angle degrees	Distance feet
1	0	4.00
2	27	25.00
3	0	

Number of lines that define the ground surface in front of the toe: 1

No.	Angle degrees	Distance feet
1	0	

Soil Layers:

Number of Layers: 3

Layers Below the Top Layer:

Coordinates of the Top of the Layer: feet

Layer	Point 1 Distance	Point 1 Elevation	Point 2 Distance	Point 2 Elevation
2	10.00	780.50	30.00	780.50
3	-40.00	770.00	30.00	770.00

Ground Water:

Include Ground Water: Yes
Phreatic Correction: Yes
Number of Points: 1

No.	Distance feet	Elevation feet
1		771.50

Soil Nails

Dimensions and Properties:

Maximum Vertical Spacing: 3.00 feet
Number of Soil Nail Rows: 4
Soil Nail Design Parameters: Uniform Throughout Cross-Section
Soil Nail Length: 30.00 feet
Inclination From Horizontal: 15 degrees
Vertical Distance from Top of Wall
to First Row: 3.50 feet
Vertical Spacing: 3.00 feet
Horizontal Spacing H: 5.00 feet
Nail Bar Diameter Ø: 1.000 inches
Nail Bar Yield Strength fy: 60.0 ksi

Facing Resistance:

	Temporary	Permanent	Seismic
LRFD Factored Facing Resistance:	27.2	36.8	36.8 kips

Soil Properties

Layer	Description	Unit Weight γ pcf	Friction Angle φ' degrees	Cohesion c' psf
1	Embankment Fill	125	24.0	100
2	V. Stiff Till	130	26.0	150
3	Shale	135	32.0	4000

Loads

Applied Loads:

Seismic:

Horizontal Seismic Coefficient Kh:

External Load:

Apply external load: No

Surcharges:

Apply surcharges: Yes

No.	Distance from Top of Wall Begin feet	End feet	Load Begin psf	Load End psf
1	27.00	55.00	250	250

Load and Resistance Factors

Load Factors:

Apply Load Factors to: Soil Nail Tensile Force (FHWA GEC No. 7 2015)

	Temporary	Permanent	Seismic
Soil Nail Tensile Force:	1.35	1.35	1.00

Resistance Factors:

	Temporary	Permanent	Seismic
Pullout (Distal):	0.65	0.65	0.65
Pullout (Proximal):	0.65	0.65	0.65
Nail Bar Yield:	0.75	0.75	0.75
Cohesion:	0.75	0.65	0.90
Friction Angle:	0.75	0.65	0.90

Search Options

Search Limits:

Begin: 3.00 feet
End: 55.00 feet

Below Toe Searches (BTS):

Perform below Toe Search: Yes
Number of BTS Points: 5
BTS Depth: 7.00 feet
Interface Friction
Reduction Factor: 0.33

Advanced Search Options:

Use Advanced Search Options: Yes
Inclination of Interslice Force: Use Average Failure Angle

Results

Analysis:

Method: LRFD
Scenario: Permanent

Capacity/Demand Ratio:

Minimum: 1.01
Found at Search Point: 10
Found at Grid Point: 11
Found at Search Level: 5.60 feet below the toe of the wall

Load at Soil Nail Head:

Calculated Service Load at Soil Nail Head (Empirical), To: 14.2 kips
Load Factor x To = To_factored: 19.1 kips
Factored Facing Resistance, F_factored (Entered): 36.8 kips
F_factored ≥ To_factored OK

Nominal Pullout Resistance:

Layer	Description	Nominal Pullout Resistance klf
1	Embankment Fill	2.413
2	V. Stiff Till	3.016
3	Shale	5.429

Results by Search Level:

** Indicates Minimum Capacity/Demand Ratio:

Search Level: At the toe of the wall Facing Design Force = 18.7 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Capacity Demand Ratio	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	1.09	3.00	29.05	3.09	88.73	13.50	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
2	1.85	8.20	34.87	3.00	69.59	16.46	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield

3	2.60	13.40	36.44	6.66	63.08	17.76	1	32.2	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
4	2.19	18.60	0.00	9.30	67.49	24.29	1	25.2	Pullout
							2	31.2	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
5	2.02	23.80	0.00	7.14	56.41	30.12	1	25.4	Pullout
							2	32.3	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
6	1.63	29.00	0.00	8.70	52.39	33.26	1	22.3	Pullout
							2	29.5	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
7	1.42	34.20	0.00	6.84	43.92	37.99	1	22.1	Pullout
							2	30.1	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
8	1.33	39.40	0.00	7.88	39.90	41.08	1	19.5	Pullout
							2	28.0	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
9	1.29	44.60	0.00	8.92	36.45	44.36	1	17.1	Pullout
							2	25.9	Pullout
							3	32.5	Pullout
							4	33.3	Bar Yield
10	1.34	49.80	0.00	14.94	37.09	43.70	1	10.5	Pullout
							2	18.5	Pullout
							3	24.1	Pullout
							4	33.3	Bar Yield
11	1.34	55.00	0.00	16.50	34.39	46.65	1	8.0	Pullout
							2	15.7	Pullout
							3	21.7	Pullout
							4	33.3	Bar Yield

Search Level: 1.40 feet below the toe of the wall Facing Design Force = 18.7 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Capacity Demand Ratio	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	1.08	3.00	57.37	3.89	86.08	13.15	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
2	1.49	8.20	20.65	5.26	78.88	17.01	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
3	1.60	13.40	21.57	5.76	67.14	20.70	1	31.0	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
4	1.56	18.60	23.13	6.07	58.75	25.10	1	28.9	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
5	1.71	23.80	41.68	31.87	-90.00	5.30	1	27.6	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
6	1.47	29.00	40.38	34.26	62.41	6.26	1	26.9	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield

7	1.30	34.20	0.00	6.84	45.41	38.97	1	20.9	Pullout
							2	28.8	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
8	1.20	39.40	0.00	11.82	45.18	39.12	1	14.8	Pullout
							2	22.7	Pullout
							3	27.2	Pullout
							4	31.7	Pullout
9	1.16	44.60	0.00	13.38	41.63	41.77	1	11.8	Pullout
							2	19.4	Pullout
							3	24.4	Pullout
							4	29.3	Pullout
10	1.18	49.80	0.00	9.96	34.86	48.55	1	13.1	Pullout
							2	22.1	Pullout
							3	28.0	Pullout
							4	33.3	Bar Yield
11	1.19	55.00	0.00	11.00	32.24	52.02	1	10.8	Pullout
							2	19.6	Pullout
							3	26.0	Pullout
							4	32.4	Pullout

Search Level: 2.80 feet below the toe of the wall Facing Design Force = 18.7 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Capacity Demand Ratio	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	1.11	3.00	52.82	4.47	88.79	14.24	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
2	1.29	8.20	44.21	5.72	75.59	16.47	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
3	1.54	13.40	22.85	5.82	68.42	21.86	1	30.6	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
4	1.45	18.60	18.74	7.86	63.84	25.31	1	26.7	Pullout
							2	33.0	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
5	1.33	23.80	21.34	7.67	56.43	30.13	1	25.4	Pullout
							2	32.3	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
6	1.19	29.00	18.52	9.18	52.27	33.17	1	22.4	Pullout
							2	29.6	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
7	1.12	34.20	23.08	7.44	43.80	37.91	1	22.2	Pullout
							2	30.2	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
8	1.09	39.40	20.30	8.40	39.77	41.01	1	19.6	Pullout
							2	28.1	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
9	1.11	44.60	18.10	9.38	36.33	44.29	1	17.2	Pullout
							2	26.1	Pullout
							3	32.7	Pullout
							4	33.3	Bar Yield
10	1.11	49.80	0.00	14.94	39.90	45.44	1	7.5	Pullout
							2	14.2	Pullout
							3	19.4	Pullout
							4	24.6	Pullout

11	1.11	55.00	0.00	16.50	37.13	48.29	1	4.6	Pullout
							2	11.0	Pullout
							3	16.6	Pullout
							4	22.2	Pullout

Search Level: 4.20 feet below the toe of the wall Facing Design Force = 16.7 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Capacity Demand Ratio	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	1.17	3.00	64.89	6.36	88.72	13.44	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
2	1.26	8.20	33.05	7.83	84.51	17.15	1	33.2	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
3	1.29	13.40	35.61	8.24	70.76	20.33	1	29.9	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
4	1.42	18.60	25.52	6.18	61.50	27.28	1	27.7	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
5	1.30	23.80	22.30	7.72	57.71	31.18	1	24.7	Pullout
							2	31.5	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
6	1.17	29.00	19.35	9.22	53.56	34.18	1	21.6	Pullout
							2	28.7	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
7	1.08	34.20	16.58	10.71	48.95	36.46	1	18.2	Pullout
							2	25.7	Pullout
							3	30.6	Pullout
							4	33.3	Bar Yield
8	1.03	39.40	14.49	12.21	44.91	38.94	1	15.0	Pullout
							2	23.0	Pullout
							3	27.6	Pullout
							4	32.1	Pullout
9	1.02	44.60	12.86	13.72	41.37	41.60	1	12.0	Pullout
							2	19.8	Pullout
							3	24.8	Pullout
							4	29.8	Pullout
10	1.05	49.80	17.05	10.42	34.61	48.41	1	13.4	Pullout
							2	22.5	Pullout
							3	28.5	Pullout
							4	33.3	Bar Yield
11	1.07	55.00	10.49	16.78	35.53	47.31	1	6.5	Pullout
							2	13.7	Pullout
							3	19.5	Pullout
							4	27.2	Pullout

Search Level: 5.60 feet below the toe of the wall Facing Design Force = 14.2 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Capacity Demand Ratio	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	1.31	3.00	66.40	6.74	88.81	14.42	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
2	1.30	8.20	50.95	11.71	86.56	13.67	1	32.8	Pullout

							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
3	1.33	13.40	32.28	9.51	75.22	21.01	1	28.6	Pullout
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
4	1.27	18.60	37.01	9.32	63.55	25.05	1	26.8	Pullout
							2	33.1	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
5	1.24	23.80	40.68	9.42	55.84	29.67	1	25.7	Pullout
							2	32.6	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
6	1.18	29.00	28.85	6.62	51.10	36.95	1	23.2	Pullout
							2	30.5	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
7	1.08	34.20	25.04	15.10	51.24	32.78	1	16.5	Pullout
							2	23.9	Pullout
							3	28.0	Pullout
							4	33.3	Bar Yield
8	1.03	39.40	15.13	12.24	46.20	39.84	1	14.0	Pullout
							2	21.5	Pullout
							3	25.9	Pullout
							4	30.3	Pullout
9	1.01	44.60	13.43	13.76	42.65	42.44	1	10.8	Pullout
							2	18.1	Pullout
							3	22.9	Pullout
							4	27.7	Pullout
** 10	1.01	49.80	12.07	15.28	39.52	45.19	1	7.9	Pullout
							2	14.8	Pullout
							3	20.0	Pullout
							4	25.3	Pullout
11	1.01	55.00	10.96	16.81	36.76	48.05	1	5.0	Pullout
							2	11.7	Pullout
							3	17.3	Pullout
							4	23.0	Pullout

Search Level: 7.00 feet below the toe of the wall Facing Design Force = 15.9 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Capacity Demand Ratio	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	2.70	3.00	82.24	22.20	0.00	0.00	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
2	4.54	8.20	0.00	2.46	76.63	24.81	1	33.3	Bar Yield
							2	33.3	Bar Yield
							3	33.3	Bar Yield
							4	33.3	Bar Yield
3	3.18	13.40	12.52	12.35	86.82	24.15	1	25.1	Pullout
							2	29.6	Pullout
							3	31.7	Pullout
							4	32.0	Pullout
4	2.33	18.60	14.78	11.54	74.32	27.52	1	22.5	Pullout
							2	28.0	Pullout
							3	30.8	Pullout
							4	32.3	Pullout
5	1.65	23.80	47.17	35.01	-90.00	6.42	1	23.6	Pullout
							2	31.3	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
6	1.46	29.00	37.48	10.96	52.73	33.52	1	22.1	Pullout

							2	29.3	Pullout
							3	33.3	Bar Yield
							4	33.3	Bar Yield
7	1.33	34.20	33.03	12.24	48.10	35.85	1	18.8	Pullout
							2	26.4	Pullout
							3	31.6	Pullout
							4	33.3	Bar Yield
8	1.28	39.40	22.94	17.11	48.46	35.65	1	12.1	Pullout
							2	19.0	Pullout
							3	23.1	Pullout
							4	29.2	Pullout
9	1.24	44.60	20.50	19.05	44.91	37.79	1	8.8	Pullout
							2	15.2	Pullout
							3	19.7	Pullout
							4	27.1	Pullout
10	1.21	49.80	18.51	21.01	41.76	40.06	1	5.5	Pullout
							2	11.6	Pullout
							3	16.5	Pullout
							4	25.3	Pullout
11	1.21	55.00	22.01	17.80	34.72	46.84	1	7.5	Pullout
							2	15.1	Pullout
							3	21.1	Pullout
							4	28.4	Pullout

END OF REPORT

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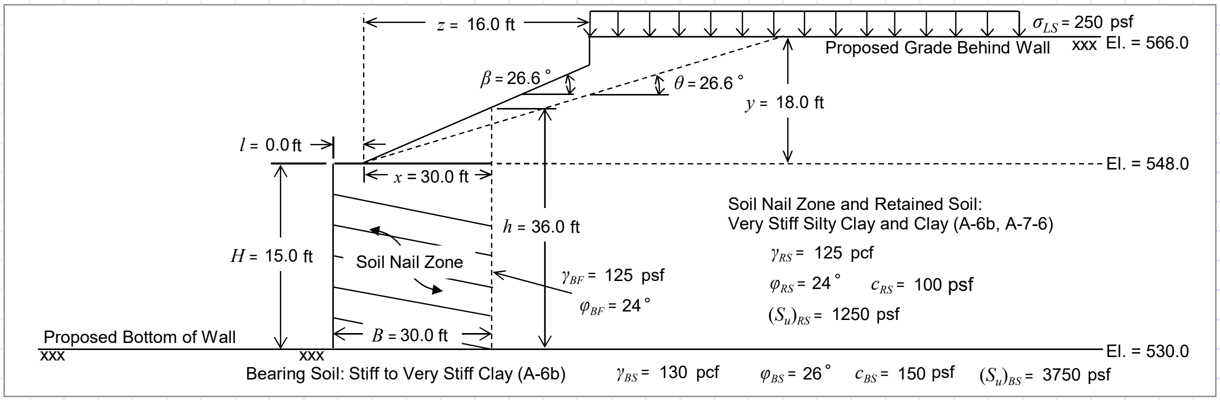
Retaining Wall 1 - Broken Backslope - 15.0 ft. Wall Height - B-001-0-19



Wall Dimensions and Retained Soil Parameters

Wall Height, (H) =	15.0	ft
Wall Width (Reinforcement Length), (B) =	30.0	ft
Distance from Wall Face to Toe of Backslope, (l) =	0.0	ft
Wall Length, (L) =	236	ft
Wall Effective Height, (h) =	36.0	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	26.6	°
Distance from Toe to Top of Backslope, (z) =	16.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	24	°
Retained Soil Drained Cohesion, (c_{RS}) =	100	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	1250	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.422	
Live Surcharge Load, (σ_{LS}) =	250	psf

Soil Nail Zone and Bearing Soil Properties:

Soil Nail Zone Unit Weight, (γ_{BF}) =	125	pcf
Soil Nail Zone Friction Angle, (φ_{BF}) =	24	°
Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	150	psf
Bearing Soil Undrained Shear Strength, $[(S_u)_{BS}]$ =	3750	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	5.0	ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

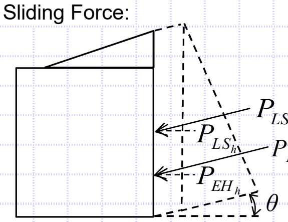
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Sections 11.10.5.3 and FHWA GES 7 Section 5.7.3

Sliding Force:

$$P_H = (P_{EH} + P_{LS})\cos\theta$$



$P_{EH} = \frac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}$ = ½(125 pcf)(36.0 ft)²(0.422)(1.50) = 51.24 kip/ft

$P_{LS} = \sigma_{LS} h K_a \gamma_{LS}$ = (250 psf)(36.0 ft)(0.422)(1.75) = 5.94 kip/ft

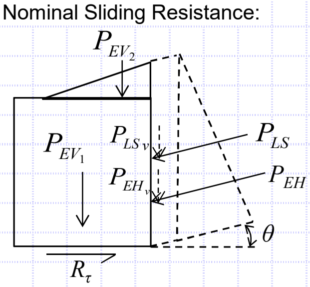
P_H = (51.24 kip/ft + 5.94 kip/ft)cos(26.6°) = 51.13 kip/ft

Check Sliding Resistance - Drained Condition

Nominal Sliding Resistance:

$$R_\tau = (P_{EV_1} + P_{EV_2} + P_{EH}\sin\theta)\tan\delta$$

(Neglect P_{LS_v} for conservatism)



$P_{EV_1} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (125 pcf)(15.0 ft)(30.0 ft)(1.00) = 56.25 kip/ft

$P_{EV_2} = \frac{1}{2}\gamma_{RS}(h-H)(B-l)\gamma_{EV}$ = ½(125 pcf)(36.0 ft - 15.0 ft)(30.0 ft - 0.0 ft)(1.00) = 39.38 kip/ft

$P_{EH} = \frac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}$ = ½(125 pcf)(36.0 ft)²(0.422)(1.50) = 51.24 kip/ft

$\tan\delta = (\tan\varphi_{BS} \le \tan\varphi_{BF}) \longrightarrow$ tan(26) ≤ tan(24) ⟶ 0.49 ≤ 0.45 = 0.45

R_τ = [56.25 kip/ft + 39.38 kip/ft + (51.24 kip/ft)sin(26.6°)](0.45) = 53.36 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \le R_\tau \cdot \phi_\tau \longrightarrow$ 51.13 kip/ft ≤ (53.36 kip/ft)(1.0) = 53.36 kip/ft ⟶ 51.13 kip/ft ≤ 53.36 kip/ft **OK**

Use φ_τ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

Retaining Wall 1 - Broken Backslope - 15.0 ft. Wall Height - B-001-0-1

Wall Height, (H) =	15.0	ft
Wall Width (Reinforcement Length), (B) =	30.0	ft
Distance from Wall Face to Toe of Backslope, (l) =	0.0	ft
Wall Length, (L) =	236	ft
Wall Effective Height, (h) =	36.0	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	26.6	°
Distance from Toe to Top of Backslope, (z) =	16.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	24	°
Retained Soil Drained Cohesion, (c_{RS}) =	100	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	1250	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.422	
Live Surcharge Load, (σ_{LS}) =	250	psf

Bearing Soil Properties:

Backfill Unit Weight, (γ_{BF}) =	125	pcf
Backfill Friction Angle, (φ_{BF}) =	24	°
Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	150	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	3750	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	5.0	ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

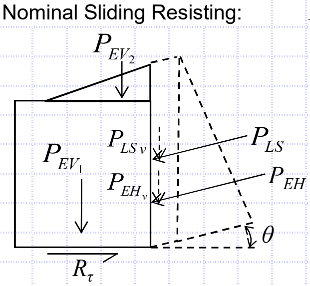
(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Sections 11.10.5.3 and FHWA GES 7 Section 5.7.3 ***(Continued)***

Check Sliding Resistance - Undrained Condition

Nominal Sliding Resisting: $R_\tau = \left((S_u)_{BS} \le \frac{q_s}{2}\right) \cdot B$



$(S_u)_{BS}$ = 3.75 ksf

$q_s = P_V / B$

$P_V = P_{EV_1} + P_{EV_2} + P_{EH} \sin\theta$

$P_{EV_1} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV}$ = (125 pcf)(15.0 ft)(30.0 ft)(1.00) = 56.25 kip/ft

(Neglect P_{LS_v} for conservatism)

$P_{EV_2} = \frac{1}{2}\gamma_{RS}(h-H)(B-l)\gamma_{EV}$ = ½(125 pcf)(36.0 ft - 15.0 ft)(30.0 ft - 0.0 ft)(1.00) = 39.38 kip/ft

$P_{EH} = \frac{1}{2}\gamma_{RS} h^2 K_a \gamma_{EH}$ = ½(125 pcf)(36.0 ft)²(0.422)(1.50) = 51.24 kip/ft

P_V = 56.25 kip/ft + 39.38 kip/ft + (51.24 kip/ft)sin(26.6°) = 118.57 kip/ft

q_s = (118.57 kip/ft) / (30 ft) = 3.95 ksf

R_τ = [3.75 ksf ≤ (3.95 ksf)/2](30.0 ft) = [3.75 ksf ≤ 1.98 ksf](30.0 ft)

R_τ = (1.98 ksf)(30.0 ft) = 59.25 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le R_\tau \cdot \phi_\tau$ ⟶ 51.13 kip/ft ≤ (59.25)(1.0) = 59.25 kip/ft ⟶ 51.13 kip/ft ≤ 59.25 kip/ft **OK**

Use φ_τ = **1.0** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

Retaining Wall 1 - Broken Backslope - 15.0 ft. Wall Height - B-001-0-1

Wall Height, (H) =	15.0	ft
Wall Width (Reinforcement Length), (B) =	36.0	ft
Distance from Wall Face to Toe of Backslope, (l) =	0.0	ft
Wall Length, (L) =	236	ft
Wall Effective Height, (h) =	36.0	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	26.6	°
Distance from Toe to Top of Backslope, (z) =	16.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	24	°
Retained Soil Drained Cohesion, (c_{RS}) =	100	psf
Retained Soil Undrained Shear Strength, [$(S_u)_{RS}$] =	1250	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.422	
Live Surcharge Load, (σ_{LS}) =	250	psf

Bearing Soil Properties:

Backfill Unit Weight, (γ_{BF}) =	125	pcf
Backfill Friction Angle, (φ_{BF}) =	24	°
Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	150	psf
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	3750	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	5.0	ft

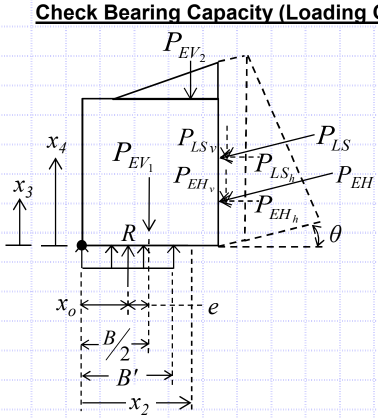
LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.10.5.4 and FHWA GES 7 Section 5.6.6



$$q_{eq} = P_V / B'$$

$$B' = B - 2e \quad = \quad 36.0\text{ ft} - 2(0.00\text{ ft}) \quad = \quad 36.00 \text{ ft}$$

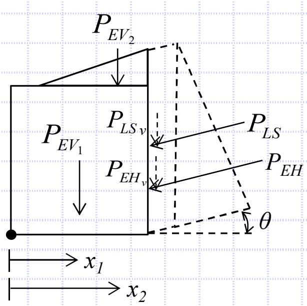
$$e = B/2 - x_o \quad = \quad (36.0\text{ ft} / 2) - 18.78\text{ ft} \quad = \quad -0.78 \text{ ft} \quad (\text{Use 0 ft})$$

$$x_o = \frac{M_V - M_H}{P_V} \quad = \quad (3997.25\text{ kip·ft/ft} - 656.67\text{ kip·ft/ft}) / 177.86\text{ kip/ft} \quad = \quad 18.78 \text{ ft}$$

$$q_{eq} = \quad (177.86\text{ kip/ft}) / (36\text{ ft}) \quad = \quad 4.94 \text{ ksf}$$

Resisting Moment, M_V:

$$M_V = P_{EV_1}(x_1) + P_{EV_2}(x_2) + P_{EH}\sin\theta(B)$$



$$P_{EV_1} = \gamma_{BF} \cdot H \cdot B \cdot \gamma_{EV} \quad = \quad (125\text{ pcf})(15.0\text{ ft})(36.0\text{ ft})(1.35) \quad = \quad 91.13 \text{ kip/ft}$$

$$P_{EV_2} = ½\gamma_{RS}(h - H)(B - l)\gamma_{EV} = ½(125\text{ pcf})(36.0\text{ ft} - 15.0\text{ ft})(36.0\text{ ft} - 0.0\text{ ft})(1.35) = 63.79 \text{ kip/ft}$$

$$P_{EH} = ½\gamma_{RS}h^2K_a\gamma_{EH} = \quad ½(125\text{ pcf})(36.0\text{ ft})^2(0.422)(1.50) \quad = \quad 51.24 \text{ kip/ft}$$

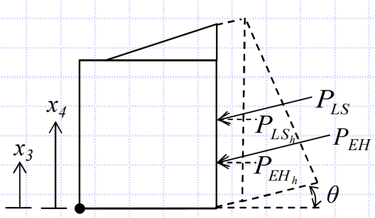
$$x_1 = B/2 \quad = \quad (36.0\text{ ft}) / 2 \quad = \quad 18.00 \text{ ft}$$

$$x_2 = l + ⅔(B - l) \quad = \quad 0.0\text{ ft} + ⅔(36.0\text{ ft} - 0.0\text{ ft}) \quad = \quad 24.00 \text{ ft}$$

$$M_V = (91.13\text{ kip/ft})(18.00\text{ ft}) + (63.79\text{ kip/ft})(24.0\text{ ft}) + (51.24\text{ kip/ft})\sin(26.6°)(36\text{ ft}) \quad = \quad 3997.25 \text{ kip·ft/ft}$$

Overturning Moment, M_H:

$$M_H = P_{EH}\cos\theta(x_3) + P_{LS}\cos\theta(x_4)$$



$$P_{EH} = ½\gamma_{RS}h^2K_a\gamma_{EH} = \quad ½(125\text{ pcf})(36.0\text{ ft})^2(0.422)(1.50) \quad = \quad 51.24 \text{ kip/ft}$$

$$P_{LS} = \sigma_{LS}hK_a\gamma_{LS} = \quad (250\text{ psf})(36.0\text{ ft})(0.422)(1.75) \quad = \quad 6.64 \text{ kip/ft}$$

$$x_3 = h/3 \quad = \quad (36.0\text{ ft}) / 3 \quad = \quad 12.00 \text{ ft}$$

$$x_4 = h/2 \quad = \quad (36.0\text{ ft}) / 2 \quad = \quad 18 \text{ ft}$$

$$M_H = (51.24\text{ kip/ft})\cos(26.6°)(12.00\text{ ft}) + (6.64\text{ kip/ft})\cos(26.6°)(18.00\text{ ft} \quad = \quad 656.67 \text{ kip·ft/ft}$$

Vertical Forces, P_V:

$$P_V = P_{EV_1} + P_{EV_2} + P_{EH}\sin\theta$$

$$P_V = 91.13\text{ kip/ft} + 63.79\text{ kip/ft} + (51.24\text{ kip/ft})\sin(26.6°) \quad = \quad 177.86 \text{ kip/ft}$$

Retaining Wall 1 - Broken Backslope - 15.0 ft. Wall Height - B-001-0-1

Wall Height, (H) =	15.0	ft
Wall Width (Reinforcement Length), (B) =	36.0	ft
Distance from Wall Face to Toe of Backslope, (l) =	0.0	ft
Wall Length, (L) =	236	ft
Wall Effective Height, (h) =	36.0	ft
Retained Soil Backslope, (β) =	26.6	°
Effective Retained Soil Backslope, (θ) =	26.6	°
Distance from Toe to Top of Backslope, (z) =	16.0	ft
Retained Soil Unit Weight, (γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	24	°
Retained Soil Drained Cohesion, (c_{RS}) =	100	psf
Retained Soil Undrained Shear Strength, $[(S_u)_{RS}]$ =	1250	psf
Retained Soil Active Earth Pressure Coeff., (K_a) =	0.422	
Live Surcharge Load, (σ_{LS}) =	250	psf

Bearing Soil Properties:

Backfill Unit Weight, (γ_{BF}) =	125	pcf
Backfill Friction Angle, (φ_{BF}) =	24	°
Bearing Soil Unit Weight, (γ_{BS}) =	130	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	26	°
Bearing Soil Drained Cohesion, (c_{BS}) =	150	psf
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	3750	psf
Embedment Depth, (D_f) =	0.0	ft
Depth to GW (Below Bot. of Wall), (D_W) =	5.0	ft

LRFD Load Factors

	EV	EH	LS
Strength Ia	1.00	1.50	1.75
Strength Ib	1.35	1.50	1.75
Service I	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

1. Drained cohesion for retained soil not accounted for in external stability analyses. This parameter is utilized in global stability analysis.

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.10.5.4 and FHWA GES 7 Section 5.6.6 *(Cor*

Check Bearing Resistance - Drained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B' N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 24.53

N_c = 22.3
s_c = 1+(36 ft/236 ft)(11.9/22.3) = 1.1
i_c = 1.0 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 13.1

N_q = 11.9
s_q = 1+(36 ft/236 ft)tan(26°) = 1.1
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(0.0 ft/36 ft)
= 1.0
i_q = 1.0 (Assumed)
C_{wq} = 5.0 ft > 0.0 ft = 1.0

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 11.3

N_γ = 12.5
s_γ = 1-0.4(36 ft/236 ft) = 0.9
i_γ = 1.0 (Assumed)
$C_{w\gamma}$ = 5.0 ft < 1.5(36 ft) + 0.0 ft = 0.5

q_n = (0 psf)(24.53) + (130 pcf)(0.0 ft)(13.1)(1.0) + ½(130 pcf)(36.0 ft)(11.3)(0.5) = 13.16 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 4.94 ksf ≤ (13.16 ksf)(0.65) = 8.55 ksf ⟶ 4.94 ksf ≤ 8.55 ksf **OK**

Use φ_b = **0.65** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

Check Bearing Resistance - Undrained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B' N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 5.14

N_c = 5.14
s_c = 1+[36 ft/(5·236 ft)] = 1.0
i_c = 1.0 (Assumed)

$N_{qm} = N_q s_q d_q i_q$ = 1.0

N_q = 1.0
s_q = 1.0
d_q = 1+2tan(26°)[1-sin(26°)]²tan⁻¹(0.0 ft/36 ft)
= 1.0
i_q = 1.0 (Assumed)
C_{wq} = 5.0 ft > 0.0 ft = 1.0

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 0.0

N_γ = 0.0
s_γ = 1.0
i_γ = 1.0 (Assumed)
$C_{w\gamma}$ = 5.0 ft < 1.5(36 ft) + 0.0 ft = 0.5

q_n = (3750 psf)(5.14) + (130 pcf)(0.0 ft)(1.0)(1.0) + ½(130 pcf)(36.0 ft)(0.0)(0.5) = 19.28 ksf

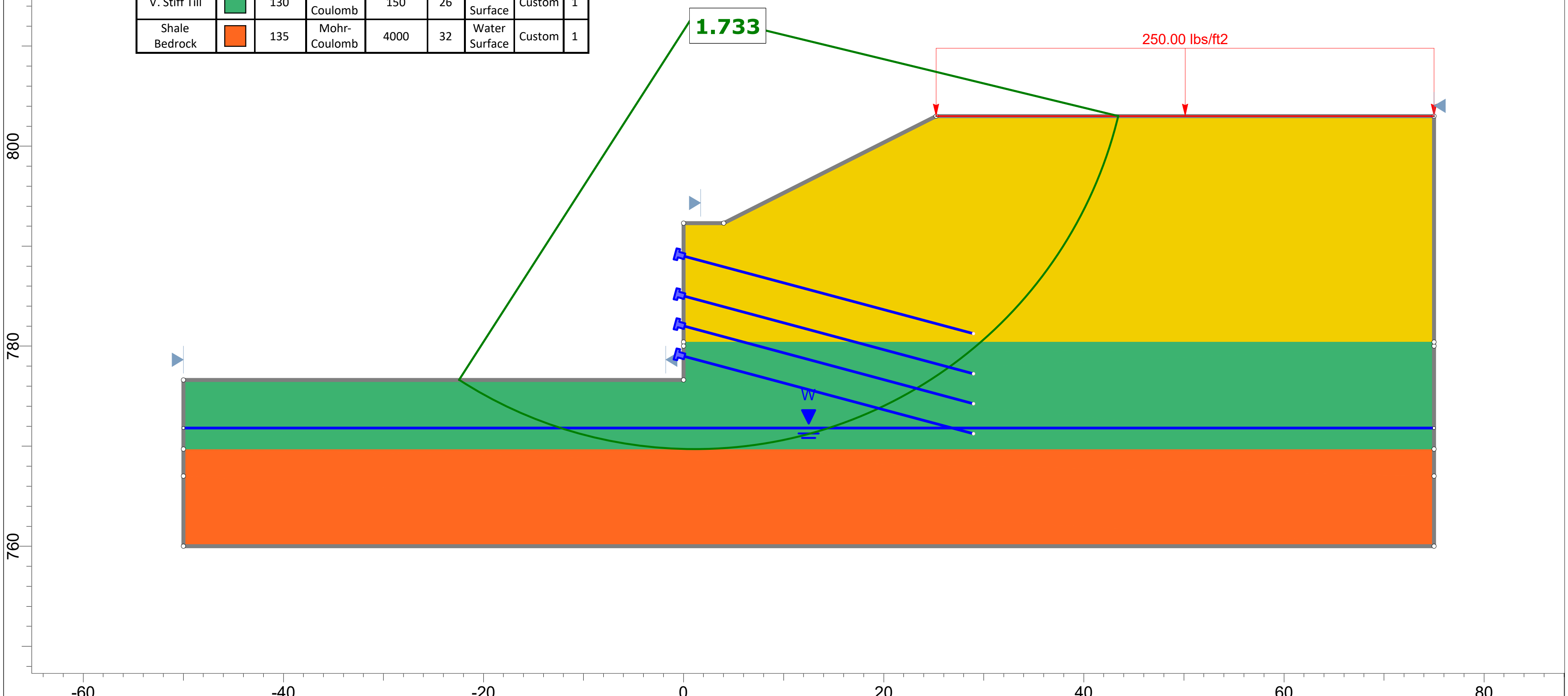
Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \le q_n \cdot \phi_b$ ⟶ 4.94 ksf ≤ (19.28 ksf)(0.65) = 12.53 ksf ⟶ 4.94 ksf ≤ 12.53 ksf **OK**

Use φ_b = **0.65** *(Per AASHTO LRFD BDM Table 11.5.6-1)*

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Embankment		125	Mohr-Coulomb	100	24	Water Surface	Custom	1
V. Stiff Till		130	Mohr-Coulomb	150	26	Water Surface	Custom	1
Shale Bedrock		135	Mohr-Coulomb	4000	32	Water Surface	Custom	1

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Support 1		Soil Nail	Active (Method A)	4.5	60000	33000	0	0	1152	Yes	Parallel to Reinforcement



SLIDEINTERPRET 9.009	*Project* SLIDE - An Interactive Slope Stability Program

Analysis	Long-Term	*Scenario*	Master Scenario
Drawn By	MDK	*Company*	Resource International, Inc.
Date	11/9/2020, 3:18:45 PM	*File Name*	FRA-70-22.85 Soil Nail Wall check_rev.slmd

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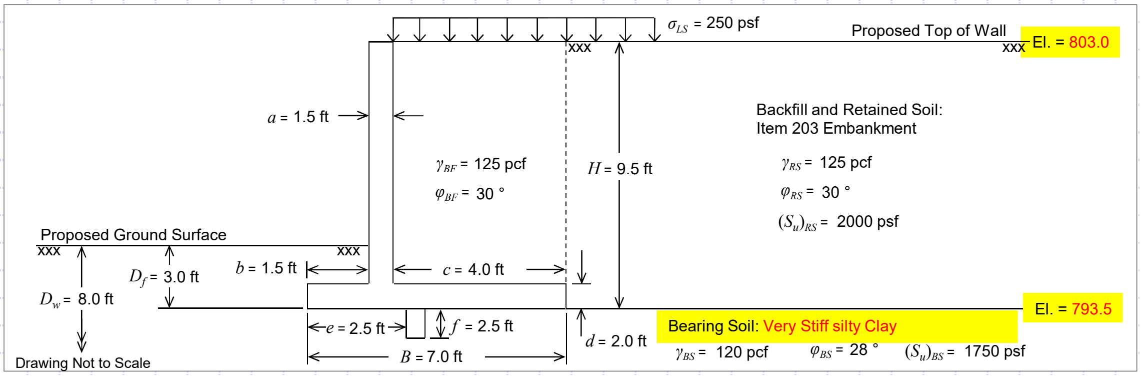
Retaining Wall 7 - CIP Wall With Shear Key - 9.5 ft. Maximum Wall Height



CIP Wall Dimensions and Surcharge Loading		
Wall Height, (H) =	9.5	ft
Foundation Width (Entire Base Width), (B) =	7.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	4.0	ft
Footing Thickness, (d) =	2.0	ft
Location of Shear Key, (e) =	2.5	ft
Depth of Shear Key, (f) =	2.5	ft
Embedment Depth, (D_f) =	3.0	ft
Wall Length, (L) =	402	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	8.0	ft

Bearing and Retained/Backfill Soil Properties:		
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	28	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1750	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	2000	psf
Active Earth Pressure Coefficient, (K_a) =	0.297	
Passive Earth Pressure Coefficient, (K_p) =	5.580	

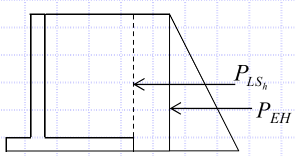
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 10.6.3.4

Sliding Force: $P_H = P_{EH} + P_{LS_h}$



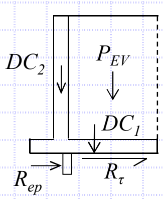
$P_{EH} = \frac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(125 pcf)(9.5 ft)²(0.297)(1.50) = 2.51 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ (250 psf)(9.5 ft)(0.297)(1.75) = 1.23 kip/ft

P_H = 2.51 kip/ft + 1.23 kip/ft = 3.74 kip/ft

Check Sliding Resistance

Nominal Sliding Resisting: $R_n = R_\tau + R_{ep}$



$$R_{ep} = \gamma_{BS} D_f f K_p \gamma_{ep} + \frac{1}{2}\gamma_{BS} f^2 K_p \gamma_{ep}$$

R_{ep} = (120 pcf)(3.0 ft)(2.5 ft)(5.58)(0.90) + ½(120 pcf)(2.5 ft)²(5.58)(0.90) = 6.40 kip/ft

Check Drained Condition: $R_\tau = P_V \tan\delta$

$$P_V = DC_1 + DC_2 + P_{EV} = \gamma_c \cdot [B \cdot d + (H - d) \cdot a] \cdot \gamma_{DC} + \gamma_{BF} \cdot (H - d) \cdot c \cdot \gamma_{EV}$$

P_V = (150 pcf) [(7.0 ft)(2.0 ft) + (9.5 ft - 2.0 ft)(1.5 ft)] (0.90) + (125 pcf)(9.5 ft - 2.0 ft)(4.0 ft)(1.00) = 7.16 kip/ft

$\tan\delta = \tan\varphi_{BS}$ = tan(28) = 0.53

R_τ = (7.16 kip/ft)(0.53) = 3.79 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Drained Condition

$P_H \le \phi_n \cdot R_n \rightarrow P_H \le \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep} \rightarrow$ 3.74 kip/ft ≤ (3.79 kip/ft)(1.00) + (6.40 kip/ft)(0.50) = 7.00 kip/ft

= 3.74 kip/ft ≤ 7.00 kip/ft **OK**

Use φ_τ = **1.00** Use φ_{ep} = **0.50** *(Per AASHTO LRFD BDM Tables 10.5.5.2.2-1 and 11.5.7-1)*

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CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	9.5	ft
Foundation Width (Entire Base Width), (B) =	7.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	4.0	ft
Footing Thickness, (d) =	2.0	ft
Location of Shear Key, (e) =	2.5	ft
Depth of Shear Key, (f) =	2.5	ft
Embedment Depth, (D_f) =	3.0	ft
Wall Length, (L) =	402	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	8.0	ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	28	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1750	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	2000	psf
Active Earth Pressure Coefficient, (K_a) =	0.297	
Passive Earth Pressure Coefficient, (K_p) =	5.580	

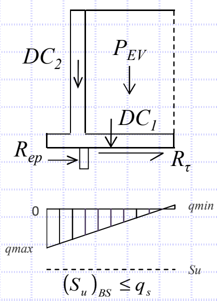
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 10.6.3.4 ***(Continued)***

Check Undrained Condition: $R_\tau = ((S_u)_{BS} \le q_s) \cdot B$



$(S_u)_{BS} =$ 1.75 ksf

$q_{max} = \frac{1}{2}\sigma_{vmax}$ = (2.26 ksf) / 2 = 1.13 ksf

$q_{min} = \frac{1}{2}\sigma_{vmin}$ = (-0.21 ksf) / 2 = -0.11 ksf

$\sigma_{vmax} = \frac{P_V}{B}\left(1 + 6\frac{e}{B}\right)$ = (7.16 kip/ft / 7.0 ft)[1 + 6(1.41 ft / 7.0 ft)] = 2.26 ksf

$\sigma_{vmin} = \frac{P_V}{B}\left(1 - 6\frac{e}{B}\right)$ = (7.16 kip/ft / 7.0 ft)[1 - 6(1.41 ft / 7.0 ft)] = -0.21 ksf

R_τ = 0.5(1.13 ksf)[((7.0 ft)(1.13 ksf)) / (1.13 ksf - -0.11 ksf)] = 3.6 kip/ft

Verify Sliding Force Less Than Factored Sliding Resistance - Undrained Condition

$P_H \le \phi_n \cdot R_n \longrightarrow P_H \le \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep} \longrightarrow$ 3.74 kip/ft ≤ (3.60 kip/ft)(1.00) + (6.40 kip/ft)(0.50) = 6.80

= 3.74 kip/ft ≤ 6.80 kip/ft **OK**

Use φ_τ = **1.00** Use φ_{ep} = **0.50** *(Per AASHTO LRFD BDM Tables 10.5.5.2.2-1 and 11.5.7-1)*

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CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	9.5	ft
Foundation Width (Entire Base Width), (B) =	7.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	4.0	ft
Footing Thickness, (d) =	2.0	ft
Location of Shear Key, (e) =	2.5	ft
Depth of Shear Key, (f) =	2.5	ft
Embedment Depth, (D_f) =	3.0	ft
Wall Length, (L) =	402	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	8.0	ft

Bearing and Retained/Backfill Soil Properties:

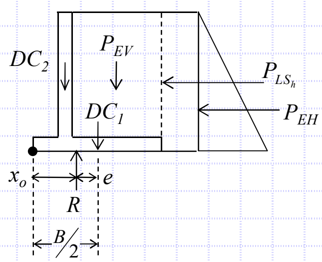
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	28	°
Bearing Soil Undrained Shear Strength, $[(s_u)_{BS}]$ =	1750	psf
Backfill and Retained Soil Unit Weight, $(\gamma_{BF}, \gamma_{RS})$ =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Undrained Shear Strength, $[(s_u)_{RS}]$ =	2000	psf
Active Earth Pressure Coefficient, (K_a) =	0.297	
Passive Earth Pressure Coefficient, (K_p) =	5.580	

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Eccentricity (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.6.3.3



$$e = B/2 - x_o$$

$$x_o = \frac{M_V - M_H}{P_V}$$ = (28.78 kip·ft/ft - 13.80 kip·ft/ft) / (7.16 kip/ft) = 2.09 ft

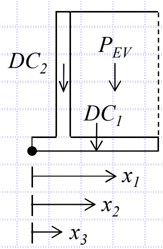
M_V = 28.78 kip·ft/ft
M_H = 13.80 kip·ft/ft
} Defined below

$P_V = P_{EV} + DC_1 + DC_2$ = 3.75 kip/ft + 1.89 kip/ft + 1.52 kip/ft = 7.16 kip/ft

$e =$ (7.0 ft / 2) - 2.09 ft = 1.41 ft

Resisting Moment, M_V:

$$M_V = P_{EV}(x_1) + DC_1(x_2) + DC_2(x_3)$$



$P_{EV} = \gamma_{BF} \cdot (H-d) \cdot c \cdot \gamma_{EV}$ = (125 pcf)(9.5 ft - 2.0 ft)(4.0 ft)(1.00) = 3.75 kip/ft

$DC_1 = \gamma_c \cdot B \cdot d \cdot \gamma_{DC}$ = (150 pcf)(7.0 ft)(2.0 ft)(0.90) = 1.89 kip/ft

$DC_2 = \gamma_c \cdot (H-d) \cdot a \cdot \gamma_{DC}$ = (150 pcf)(9.5 ft - 2.0 ft)(1.5 ft)(0.90) = 1.52 kip/ft

$x_1 = a + b + c/2$ = 1.5 ft + 1.5 ft + (4.0 ft / 2) = 5.0 ft

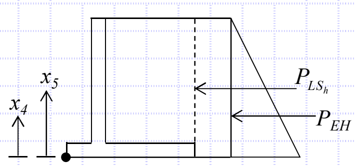
$x_2 = B/2$ = 7.0 ft / 2 = 3.5 ft

$x_3 = b + a/2$ = 1.5 ft + (1.5 ft / 2) = 2.3 ft

$M_V =$ (3.75 kip/ft)(5.0 ft) + (1.89 kip/ft)(3.5 ft) + (1.52 kip/ft)(2.3 ft) = 28.78 kip·ft/ft

Overturning Moment, M_H:

$$M_H = P_{EH}(x_2) + P_{LS_h}(x_3)$$



$P_{EH} = \frac{1}{2}\gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(125 pcf)(9.5 ft)²(0.297)(1.50) = 2.51 kip/ft

$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(9.5 ft)(0.297)(1.75) = 1.23 kip/ft

$x_2 = H/3$ = (9.5 ft) / 3 = 3.17 ft

$x_3 = H/2$ = (9.5 ft) / 2 = 4.75 ft

$M_H =$ (2.51 kip/ft)(3.17 ft) + (1.23 kip/ft)(4.75 ft) = 13.8 kip·ft/ft

Limiting Eccentricity:

$e_{max} = B/3 \longrightarrow e_{max} =$ (7.0 ft) / 3 = 2.33 ft

Check Eccentricity

$e < e_{max} \longrightarrow$ 1.41 ft < 2.33 ft OK

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	9.5	ft
Foundation Width (Entire Base Width), (B) =	7.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	4.0	ft
Footing Thickness, (d) =	2.0	ft
Location of Shear Key, (e) =	2.5	ft
Depth of Shear Key, (f) =	2.5	ft
Embedment Depth, (D_f) =	3.0	ft
Wall Length, (L) =	402	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	8.0	ft

Bearing and Retained/Backfill Soil Properties:

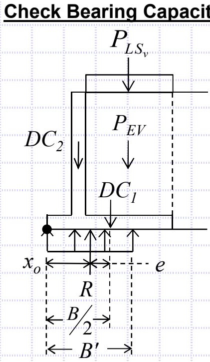
Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	28	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1750	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	2000	psf
Active Earth Pressure Coefficient, (K_a) =	0.297	
Passive Earth Pressure Coefficient, (K_p) =	5.580	

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2



$$q_{eq} = P_V / B'$$

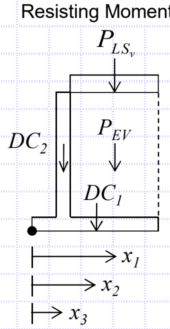
$B' = B - 2e$ = 7.0 ft - 2(0.33 ft) = 6.34 ft

$e = B/2 - x_o$ = (7.0 ft / 2) - 3.17 ft = 0.33 ft

$x_o = \dfrac{M_V - M_H}{P_V}$ = (54.56 kip·ft/ft - 13.80 kip·ft/ft) / (12.86 kip/ft) = 3.17 ft

q_{eq} = (12.86 kip/ft) / (6.34 ft) = 2.03 ksf

Resisting Moment, M_V:



$$M_V = P_{EV}(x_1) + P_{LS_v}(x_1) + DC_1(x_2) + DC_2(x_3)$$

$P_{EV} = \gamma_{BF} \cdot (H - d) \cdot c \cdot \gamma_{EV}$ = (125 pcf)(9.5 ft - 2.0 ft)(4.0 ft)(1.35) = 5.06 kip/ft

$P_{LS_v} = \sigma_{LS} \cdot B \cdot \gamma_{LS}$ = (250 psf)(7.0 ft)(1.75) = 3.063 kip/ft

$DC_1 = \gamma_c \cdot B \cdot d \cdot \gamma_{DC}$ = (150 pcf)(7.0 ft)(2.0 ft)(1.25) = 2.63 kip/ft

$DC_2 = \gamma_c \cdot (H - d) \cdot a \cdot \gamma_{DC}$ = (150 pcf)(9.5 ft - 2.0 ft)(1.5 ft)(1.25) = 2.11 kip/ft

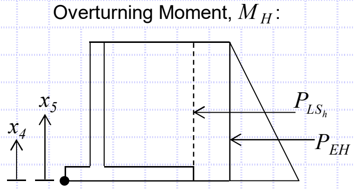
$x_1 = a + b + c/2$ = 1.5 ft + 1.5 ft + (4.0 ft / 2) = 5.0 ft

$x_2 = B/2$ = 7.0 ft / 2 = 3.5 ft

$x_3 = b + a/2$ = 1.5 ft + (1.5 ft / 2) = 2.3 ft

M_V = (5.06 kip/ft)(5.0 ft) + (3.06 kip/ft)(5.0 ft) + (2.63 kip/ft)(3.5 ft) + (2.11 kip/ft)(2.3 ft) = 54.56 kip·ft/ft

Overturning Moment, M_H:



$$M_H = P_{EH}(x_4) + P_{LS_h}(x_5)$$

$P_{EH} = \frac{1}{2} \gamma_{RS} H^2 K_a \gamma_{EH}$ = ½(125 pcf)(9.5 ft)²(0.297)(1.50) = 2.51 kip/ft

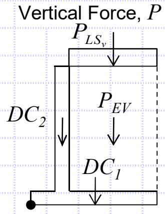
$P_{LS_h} = \sigma_{LS} H K_a \gamma_{LS}$ = (250 psf)(9.5 ft)(0.297)(1.75) = 1.23 kip/ft

$x_4 = H/3$ = (9.5 ft) / 3 = 3.17 ft

$x_5 = H/2$ = (9.5 ft) / 2 = 4.75 ft

M_H = (2.51 kip/ft)(3.17 ft) + (1.23 kip/ft)(4.75 ft) = 13.8 kip·ft/ft

Vertical Force, P_V:



$$P_V = P_{EV} + P_{LS_v} + DC_1 + DC_2$$

P_V = 5.06 kip/ft + 3.06 kip/ft + 2.63 kip/ft + 2.11 kip/ft

P_V = 12.86 kip/ft

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	9.5	ft
Foundation Width (Entire Base Width), (B) =	7.0	ft
Stem Width, (a) =	1.5	ft
Toe Width, (b) =	1.5	ft
Heel Width, (c) =	4.0	ft
Footing Thickness, (d) =	2.0	ft
Location of Shear Key, (e) =	2.5	ft
Depth of Shear Key, (f) =	2.5	ft
Embedment Depth, (D_f) =	3.0	ft
Wall Length, (L) =	402	ft
Live Surcharge Load, (σ_{LS}) =	250	psf
Depth to Groundwater, (D_w) =	8.0	ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	120	pcf
Bearing Soil Friction Angle, (φ_{BS}) =	28	°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	1750	psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	125	pcf
Retained Soil Friction Angle, (φ_{RS}) =	30	°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	2000	psf
Active Earth Pressure Coefficient, (K_a) =	0.297	
Passive Earth Pressure Coefficient, (K_p) =	5.580	

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2 ***(Continued)***

Check Bearing Resistance - Drained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B' N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 26.035 $N_{qm} = N_q s_q d_q i_q$ = 16.796 $N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 16.617

N_c = 25.803
s_c = 1+(6.34 ft/402 ft)(14.72/25.803) = 1.009
i_c = 1.000 (Assumed)

N_q = 14.72
s_q = 1+(6.34 ft/402 ft)tan(28°) = 1.008
d_q = 1+2tan(28°)[1-sin(28°)]²tan⁻¹(3.0 ft/6.34 ft) = 1.132
i_q = 1.000 (Assumed)
C_{wq} = 8.0 ft > 3.0 ft = 1.000

N_γ = 16.717
s_γ = 1-0.4(6.34 ft/402 ft) = 0.994
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 8.0 ft < 1.5(6.34 ft) + 3.0 ft = 0.921

q_n = (0 psf)(26.035) + (120 pcf)(3.0 ft)(16.796)(1.000) + ½(120 pcf)(6.3 ft)(16.617)(0.921) = 11.87 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 2.03 ksf ≤ (11.87 ksf)(0.55) = 6.53 ksf ⟶ 2.03 ksf ≤ 6.53 ksf **OK**

Use φ_b = **0.55** *(Per AASHTO LRFD BDM Table 11.5.7-1)*

Check Bearing Resistance - Undrained Condition

Nominal Bearing Resistance: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B' N_{\gamma m} C_{w\gamma}$

$N_{cm} = N_c s_c i_c$ = 5.186 $N_{qm} = N_q s_q d_q i_q$ = 1.000 $N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ = 0.000

N_c = 5.140
s_c = 1+(6.34 ft/[(5)(402 ft)]) = 1.009
i_c = 1.000 (Assumed)

N_q = 1.000
s_q = 1.000
d_q = 1+2tan(0°)[1-sin(0°)]²tan⁻¹(3.0 ft/6.34 ft) = 1.000
i_q = 1.000 (Assumed)
C_{wq} = 8.0 ft > 3.0 ft = 1.000

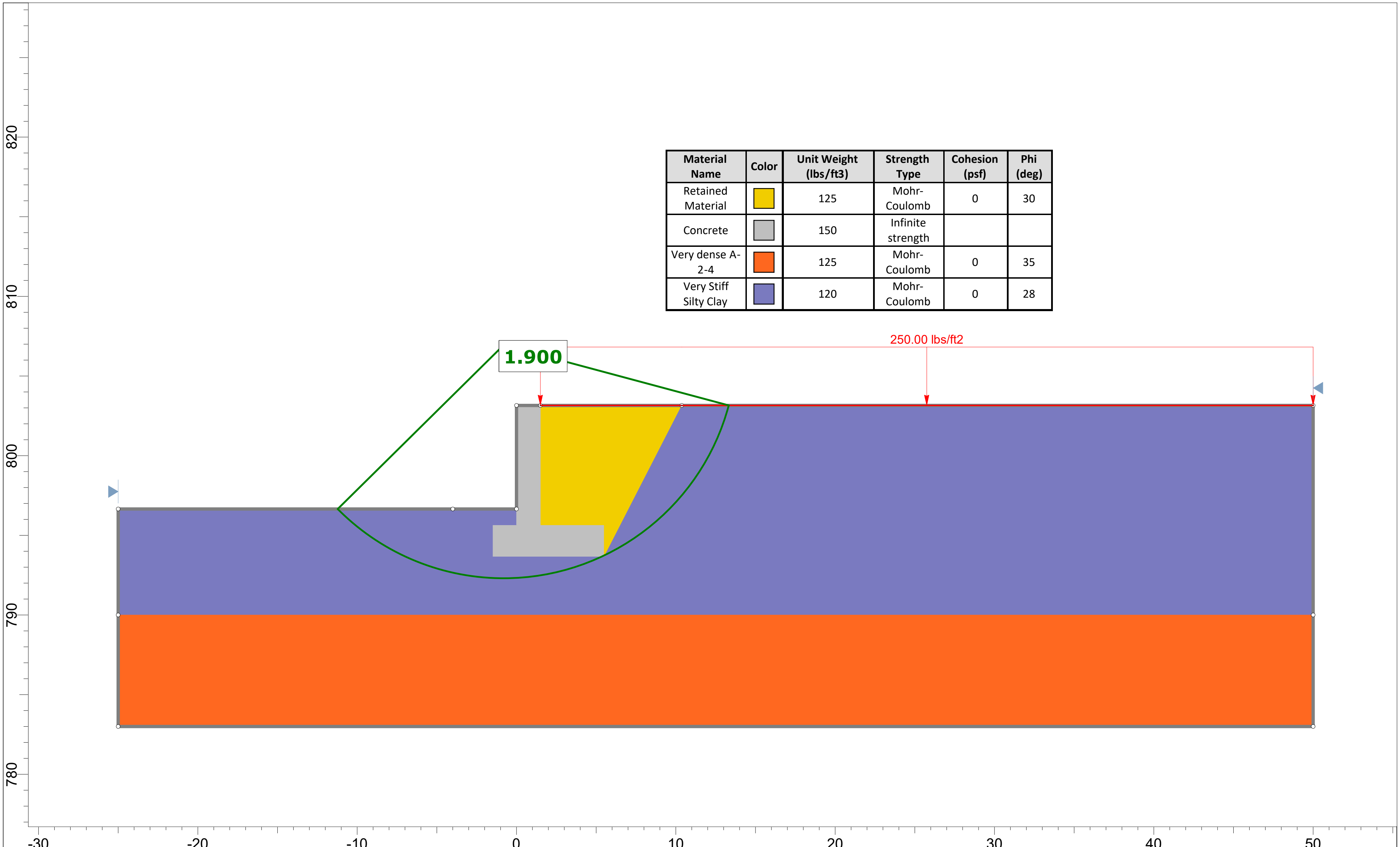
N_γ = 0.000
s_γ = 1.000
i_γ = 1.000 (Assumed)
$C_{w\gamma}$ = 8.0 ft < 1.5(6.34 ft) + 3.0 ft = 0.921

q_n = (1750 psf)(5.186) + (120 pcf)(3.0 ft)(1.000)(1.000) + ½(120 pcf)(6.3 ft)(0.000)(0.921) = 9.44 ksf

Verify Equivalent Pressure Less Than Factored Bearing Resistance

$q_{eq} \leq q_n \cdot \phi_b$ ⟶ 2.03 ksf ≤ (9.44 ksf)(0.55) = 5.19 ksf ⟶ 2.03 ksf ≤ 5.19 ksf **OK**

Use φ_b = **0.55** *(Per AASHTO LRFD BDM Table 11.5.7-1)*



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Retained Material		125	Mohr-Coulomb	0	30
Concrete		150	Infinite strength		
Very dense A-2-4		125	Mohr-Coulomb	0	35
Very Stiff Silty Clay		120	Mohr-Coulomb	0	28

Project	SLIDE - An Interactive Slope Stability Program		
Analysis	Long-Term	*Scenario*	Master Scenario
Drawn By	MDK	*Company*	Resource International, Inc.
Date	2/1/2021, 4:25:29 PM	*File Name*	Wall 7 Global Check.slmd

SLIDEINTERPRET 9.009