REVISED REPORT
STRUCTURE FOUNDATION EXPLORATION
RETAINING WALL #1 AND #4
MED-18-13.54
MEDINA COUNTY, OHIO
PID#: 92953

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NEAS PROJECT 15-0091

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EXECUTIVE SUMMARY

The Ohio Department of Transportation (ODOT) has proposed a project for improvements to State Route 18 (MED-18-13.54, PID 92953) in the City of Medina, Medina County, Ohio. The SR-18 roadway alignment is located in the Killbuck-Glaciated Pittsburgh Plateau physiographic region, which is part of the Glaciated Allegheny Plateaus. This area is characterized by ridges and flat uplands dissected by steep valleys. This topography is reflected in the steep valley of the West Branch Rocky River which crosses MED-18 midway at an elevation of about 910 feet (ft) as compared to the western and eastern ends of the alignment which rise to ~1,000 ft and 1,060, respectively. The alignment is underlain by till from 80 to 320 ft deep.

National Engineering & Architectural Services, Inc. (NEAS), formerly Barr Engineering, Inc., has been contracted to perform geotechnical engineering services for the project. The purpose of the geotechnical engineering services was to perform geotechnical explorations within the project limits to obtain information concerning the subsurface soil and groundwater conditions relevant to the design and construction of the project. The scope of work performed by NEAS as part of the referenced project included: a review of published geotechnical information; performing 3 total test borings (plus 5 project borings drilled in 2014 and 2016); laboratory testing of soil samples in accordance with the SGE; performing geotechnical engineering analyses to assess retaining wall design and construction considerations; and development of this summary report.

NEAS presents this Structure Foundation Exploration Report for the proposed construction of Retaining Wall #1 (RW#1) and Retaining Wall #4 (RW#4) for the Part 2 MED-18-13.54 Improvements to State Route 18 (SR-18) within the City of Medina, Ohio. The proposed Retaining Wall #1 will be a modular block wall along the eastbound of SR-18 from STA 126+89.28 to STA 129+05.28. For much of the length of the wall along SR-18, the roadside face of the wall will be offset 52 ft from the centerline of construction of SR-18. The proposed wall is anticipated to be approximately 216 ft in length and will have a maximum exposed wall height of approximately 6.5 ft. The proposed Retaining Wall #4 is located approximately 52 ft right of SR-18 alignment from approximate STA 167+40 with a curve that extends south along River Styx Rd to approximate STA 920+63. RW#4 is proposed as a soldier pile wall by GPD group. It is anticipated to be approximately 336 ft in length and will have a maximum total height of 9.3 ft.

The subsurface profile within the proposed project area consists of surficial materials comprised of granular base, asphalt pavement section, brick or topsoil underlain by “man-made” fill comprised of mixtures of silt, clay and gravel and stone fragments atop glacial tills. Bedrock was not encountered within depths of the borings performed.

In this report, geotechnical analyses consisting of bearing resistance for RW#1, external stability (i.e., overall stability, and maximum moment) for RW#4, global stability and seismic analysis for the project site were performed. The factored bearing resistance for various block widths (1ft, 2ft and 3ft) of RW#1 was estimated for both effective stress and total stress conditions, which is between 1.2 ksf and 4.0 ksf. According to the global stability analyses, the minimum slope stability factor for both short-term (Total Stress) and long-term (Effective Stress) conditions exceeded the desire value of 1.54.

Factored maximum moments for both effective stress and total stress analysis were determined utilizing the provided RW#4 sections in our analyses. Capacity to demand ratios (CDR) for overall stability were calculated at the Strength Limit State. Based on our analyses, it is recommended that 30-inch diameter drilled shafts at 8 ft spacing with HP 14x73 are utilized as solider pile and the embedment depth of drilled shaft below the design grade is 16 ft. It is our opinion that the subsurface conditions encountered are generally satisfactory and will provide adequate resistance assuming the proposed RW#1 and RW#4 are constructed in accordance with the recommendations provided within this report, as well as all applicable standards and specifications (i.e., ODOT, manufacture, etc.) for modular block wall and soldier pile wall construction.

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1. INTRODUCTION

1.1. General

NEAS, formerly Barr Engineering, Inc. (BEI), presents our Structure Foundation Exploration Report for the proposed construction of Retaining Wall #1 (RW#1) along State Route 18 (SR-18) starting from STA 126+89.28 to STA 129+05.28, and Retaining Wall #4 (RW#4) along State Route 18 (SR-18) and River Styx Road (Rd) in the southwest quadrant of the intersection of the two referenced roadways at Goodwill Building side hill cut section. Both walls are proposed as part of the SR-18 widening and improvement project (MED-18-13.54, PID 92953) in the City of Medina, Medina County, Ohio.

This report presents a summary of the encountered superficial and subsurface conditions and our recommendations for retaining wall foundation design and construction in accordance with Load and Resistance Factor Design (LRFD) method as set forth in AASHTO's Publication LRFD Bridge Design Specifications, 7th Edition with 2015 interim revisions (BDS) (AASHTO, 2014) and ODOT's 2007 LRFD Bridge Design Manual (BDM) (ODOT, 2007).

MED-18-13.54 Phase 3 project consists of:

- Culverts #1, #3 and #6 with full height headwalls;
- Retaining Wall #1 at SR-18 stationing 126+89.28;
- Retaining Wall #4 Goodwill Building side hill cut section.

The exploration was conducted in general accordance with National Engineering & Architectural Services, Inc.'s (NEAS's) proposal to GPD Group (GPD), dated April 17, 2017 and with the provisions of ODOT's *Specifications for Geotechnical Explorations* (SGE) (ODOT, 2016).

The scope of work performed by NEAS as part of the referenced project included: a review of published geotechnical information; performing 3 total test borings (plus 5 project borings drilled in 2014 and 2016); laboratory testing of soil samples in accordance with the SGE; performing geotechnical engineering analysis to assess retaining wall design and construction considerations; and development of this summary report.

1.2. Proposed Construction

In order to limit the influence of this widening on the roadway Waterford Drive, RW#1 is proposed to retain the eastbound of SR-18 from STA 126+89.28 to STA 129+05.28. Based on the site plan and cross section provided by GPD Group, Inc. (GPD), dated March 1, 2018, RW#1 will be a modular block wall. For much of the length of the wall along SR-18, the roadside face of the wall will be offset 52 ft from the centerline of construction of SR-18. The proposed wall is anticipated to be approximately 216 ft in length and will have a maximum exposed wall height of approximately 7.0 ft.

In order to limit the impact that this widening and subsequent embankment construction will have into the slope of the Goodwill Industries property, an approximately 336 feet long RW#4 is required along the eastbound of SR-18 and around the intersection along the southbound of River Styx Road. The proposed RW#4 will allow for the preservation of as much of the greenspace and landscaping around the Goodwill Industries property as possible. According to the email from GPD Group dated January 4, 2016, the preferred wall type is a solider pile wall with temporary wood lagging and permanent concrete wall facing. Based on Wall4 Type Study dated December 21, 2016, the retaining wall is proposed along the south side of SR-18 from approximate STA 167+40 with a curve that extends south along River Styx Rd to approximate STA 920+63. For much of the length of the wall along SR-18, the roadside face of the wall will be offset 52 ft from the centerline of construction of SR-18. The proposed wall is anticipated to be approximately 336 ft in length and will have a maximum exposed wall height of approximately 9.3 ft.

2. GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1. Geology and Physiography

The project site is located in the Killbuck-Glaciated Pittsburgh Plateau physiographic region, which is part of the Glaciated Allegheny Plateaus (Brockman, 1998). This area is characterized by ridges and flat uplands dissected by steep valleys. This topography is reflected in the steep valley of the W Branch Rocky River which crosses MED-18 midway at an elevation of about 910 ft as compared to the western and eastern ends of the alignment which rise to ~1,000 ft and 1,060, respectively.

The project site is underlain by Wisconsinan-age till (unsorted mix of clay, silt, sand, gravel and boulders) over sandstone and shale deposited in Mississippian-age (ODNR, 2000). Bedrock topography maps indicated depth of bedrock ranging from elevation 650 ft to 1,000 ft, placing it between 50 ft and 260 ft deep (Schumacher, et al, 1996). It is mapped as Mississippian-age Cuyahoga Formation (Slucher, et al, 1996).

2.2. Hydrology/Hydrogeology

SR-18 crosses over West Branch Rocky River at approximately STA 741+00 where the flow line elevation is 910 ft and likely represents the local groundwater table. Broadway Creek, a tributary to West Branch flows under SR-18 at approximately STA 702+50 (U.S. Department of the Interior, 2013) and a tributary to Broadway flows under the alignment at ~STA 711+50.

The West Branch Rocky River and the area immediately adjacent to it are located in a special flood hazard zone subject to inundation by the 1% annual chance flood (US Department of Homeland Security, 2013). The base flood elevation where SR-18 crosses West Branch is 917 ft.

2.3. Mining and Oil/Gas Production

No abandoned mines are noted on ODNR's Abandoned Underground Mine Locator in the vicinity of the proposed retaining wall site (ODNR [1], 2016).

The following (Table 1) gas wells were noted in the vicinity of the alignment (ODNR, 2015(2)). All but three are abandoned and/or plugged. The remaining three have not produced gas since 1993.

Table 1: Gas wells in Proximity to the Alignment

Well Name	Owner	Well No.	Formation	Status	Direction from Alignment
Medina Community Hospital	Ohio Fuel Gas Co.	Well No. 8	Gas-Clinton Sand	Plugged & Abandoned	~520 ft south
	Hydrocarbon Resources LTD	MCZ#1		Not Drilled	
	Buckeye Well Surveys	1	Gas-Clinton Sand	Abandoned 1996	
J H Witzel	O.F.S	2	Gas	Plugged & Abandoned	~765 ft south
ES Johnson	Martin H Lax	1	Gas	Abandoned 1991	~380 ft north
-	-	5	Gas	Plugged & Abandoned	~601 ft south
Tru-Fit	Tru-Filt Products Corp	5, 6A	Gas	Installed 1983 - Ohio Shale – Berea Sandstone – no production since 1993	~120 ft south

2.4. Historical Records and Previous Phases of Project Exploration

The following report/plans were available for review and evaluation for this report (ODOT, 2016):

- The August 19, 2015 Draft subgrade exploration report for Project MED-18-13.54 prepared by Barr Engineering, INC. (BEI, 2015).

2.5. Site Reconnaissance

The site reconnaissance was conducted on July 16, 2015 and August 7, 2015. RW#1 is between SR-18 eastbound and Waterford Dr (Photograph 1). The embankment slope is thickly vegetated with a grassed yard at the foot. Culvert #3 runs from the southernmost point of the lake under SR-18 to connect with a pond on the opposite side of the road. Several of the older trees in the area are also slanted downslope slightly, while the newer trees are slanted slightly more upslope (Photograph 1). Several sources of water flow into and out of this area, including 1 culvert, a lake, the roadway, and potential overflow of the pond, making high water levels during heavy rainfall extremely likely.

The existing embankment slope at the proposed wall #4 location appeared to have an estimated average slope of about 4 Horizontal to 1 Vertical (4H:1V). The slope is heavily vegetated, and appeared to be in good condition with no signs of instability observed. Furthermore, the existing vegetation (i.e., trees, saplings, etc.) was observed be vertical on the side slopes (see Photograph 2).

Photograph 1: View of Waterford Dr. at RW#1 site

Photograph 2: View of south of SR-18 at RW#4 site

3. GEOTECHNICAL EXPLORATION

3.1. Field Exploration Program

The original exploration for RW#1 was conducted by NEAS between July 2, 2015 and July 8, 2015 and included 2 borings both drilled to depths 30.0 ft bgs to 31.5 ft bgs. The additional exploration for RW#1 was conducted between September 20, 2016 and September 22, 2016 and included 3 borings drilled to depths 51.5 ft bgs to 61.5 ft bgs.

The original exploration for #4 was conducted by NEAS on September 15, 2016 and included 2 borings both drilled to depths 21.5 ft bgs. The additional exploration for RW#4 was conducted between December 6, 2017 and December 14, 2017 and included 3 borings all drilled to depths 46.5 bgs.

The boring locations were selected by NEAS in general accordance with the guidelines contained in the SGE with the intent to evaluate subsurface soil and groundwater conditions. The borings were typically located along/near the proposed wall alignment in locations that were not restricted by maintenance of traffic, underground utilities or dictated by terrain (i.e. steep embankment slopes).

Stationing, offset, elevations and latitude and longitude locations of the project borings are provided in Table 2 below.

Table 2: Project Boring Summary

Boring Number	Location (Sta/Offset)(2)	Latitude(1)	Longitude(1)	Elevation (NAVD 88) (ft)	Depth (ft)
Retaining Wall #1					
B-015-0-14	127+12, 23' LT	41.137745	-81.827227	959.6	31.5
B-016-0-14	127+42, 19' RT	41.137619	-81.827141	959.9	30.0
B-014-1-16	126+68, 27' LT	41.137761	-81.827385	959.2	61.5
B-015-1-16	127+62, 22' LT	41.137721	-81.827046	960.0	61.5
B-016-1-16	127+78, 18' RT	41.137607	-81.827014	960.9	51.5
Retaining Wall #4					
B-027-3-16	168+47, 19' RT	41.136296	-81.812359	968.0	21.5
B-027-4-16	168+08, 89' RT	41.136101	-81.812483	979.1	21.5
B-027-5-17	167+39, 71' RT	41.136136	-81.812728	974.2	46.5
B-027-6-17	169+93, 69' RT	41.136148	-81.811841	982.0	46.5
B-027-7-17	920+68, 36' LT	41.135915	-81.811858	982.4	46.5

Notes:
1. *As-drilled boring location and corresponding ground surface elevation were surveyed in the field by GPD Group.*
2. *Stationing in reference to centerline of Construction SR-18 or River Styx Rd.*

The boring was drilled using a CME 45B, CME 55 or CME 55X truck mounted drilling rig utilizing 3.25-inch diameter hollow stem augers. In general, soil samples were recovered at intervals of 2.5-ft to a depth of 20 ft bgs and at 5.0-ft intervals thereafter using a split spoon sampler (AASHTO T-206 "Standard Method for Penetration Test and Split Barrel Sampling of Soils."). Borings drilled as dual-purpose borings for both structure foundation and roadway subgrade characterization purposes obtained samples continuously within depths of the borings corresponding to the subgrade elevation of the proposed roadway grades. The soil samples obtained from the exploration program were visually observed in the field by the NEAS field representative and preserved for review by a Geologist and possible laboratory testing. Standard penetration tests (SPT) were conducted using a CME auto hammer that has been calibrated to be 77.4%, 81.8% and 85.4% efficient as indicated on the boring logs.

Field boring log was prepared by drilling personnel, and included lithological description, SPT results recorded as blows per 6-inch increment of penetration, and estimated unconfined shear strength values on specimens exhibiting cohesion (using a hand-penetrometer). Groundwater measurements were attempted during the boring drilling procedures and immediately following the completion of the borehole. After completing the boring, the borehole was backfilled with auger cuttings and asphalt patch following SGE section 407.

3.2. Laboratory Testing Program

The laboratory testing program consisted of classification testing and moisture content determinations. Data from the laboratory-testing program were incorporated onto the borings logs (Appendix B). Soil samples are retained at the laboratory for 60 days following report submittal, after which time they will be discarded.

3.2.1. Classification Testing

Representative soil samples were selected for index properties (Atterberg Limits) and gradation testing for classification purposes on approximately 31% of the samples. At the boring location, samples were selected for testing with the intent of identification and classification of all significant soil units. Soils not selected for testing were compared to laboratory tested samples/strata and classified visually. Moisture content testing was conducted on all samples. The laboratory testing was performed in general accordance with applicable AASHTO specifications.

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

3.2.2. Standard Penetration Test Results

Standard Penetration Tests (SPT) and split-barrel (commonly known as split-spoon) sampling of soils were performed at varying intervals (i.e., 2.5-ft or 5.0-ft intervals) in the project borings performed. To account for the high efficiency (automatic) hammers used during SPT sampling, field SPT N-values were converted based on the calibrated efficiency (energy ratio) of the specific drill rig's hammer. Field N-values were converted to an equivalent rod energy of 60% (N_{60}) for use in analysis or for correlation purposes. The resulting N_{60} values are presented on the boring logs provided in Appendix B.

4. GEOTECHNICAL FINDINGS

The subsurface conditions encountered during NEAS’s explorations are described in the following subsections and on the boring log presented in Appendix B. The boring log represents NEAS’s interpretation of the subsurface conditions encountered at the boring location based on our site observations, field log, visual review of the soil samples by NEAS's geologist, and laboratory test results. The lines designating the interfaces between various soil strata on the boring log represent the approximate interface location; the actual transition between strata may be gradual and indistinct. The subsurface soil and groundwater characterizations included herein, including summary test data, are based on the subsurface findings from the geotechnical explorations performed by NEAS as part of the referenced project, results of historical explorations, and consideration of the geological history of the site.

4.1. Subsurface Conditions

The subsurface profile within the proposed project area consists of surficial materials comprised of granular base, asphalt pavement section, brick or topsoil underlain by “man-made” fill comprised of mixtures of silt, clay and gravel and stone fragments atop glacial tills. Bedrock was not encountered within depths of the borings performed.

4.1.1. Overburden Soil

1) RW#1 Site

At the proposed RW#1 site, three different materials were encountered below the existing pavement section. In general, the three different overburden materials consisted of: 1) embankment "man-made" fill soils; 2) cohesive, fine-grained glacial tills; and 3) layer of silt and sandy silt. These materials and the general profile is further described below.

Soil visually identified as fill was encountered in two borings (B-015-0-14 and B-016-0-14). These fill soils were encountered immediately below the surficial materials and extended to depths ranging from 7.3 to 9.5 ft below ground surface (elevations 959.6 to 959.9 ft amsl). Based on laboratory testing results and a visual review of the fill samples obtained, the embankment fills at the site are primarily classified as Silt and Clay (A-6a), Silty Clay (A-6b) and Clay (A-7-6). With respect to the soil strength, the soils can be described as having a relative consistency ranging from soft to hard correlating to converted SPT-N values (N_{60}) between 9 and 53 blows per foot (bpf). Natural moisture contents of the fill ranged from 17% to 24% in moisture.

The soil stratum encountered immediately beneath the embankment fill generally consisted of cohesive, fine grained glacial till soils. Based on laboratory testing results and a visual review of the fill samples obtained, the glacial till soils encountered are comprised primarily of Silt and Clay(A-6a), Silty Clay (A-6b), Clay (A-7-6), Silt (A-4b) and Sandy Silt (A-4a). With respect to the soil strength, the soils can be described as having a relative consistency ranging from very soft to hard correlating to converted SPT-N values (N_{60}) between 8 blows per foot (bpf) and refusal. Natural moisture contents of the cohesive soils ranged from 11% to 33% in moisture. Specially, Weight of Hammer (WOH) soil (A-4a) was encountered in boring B-015-0-14.

2) RW#4 Site

At the proposed RW#4 site, four different materials were encountered below the existing pavement section. In general, the four different overburden materials consisted of: 1) embankment "man-made" fill soils; 2) cohesive, fine-grained glacial tills; 3) intermittent layers of coarse and fine sand; and 4) layer of silt and sandy silt. These materials and the general profile is further described below.

Soil visually identified as fill was encountered in all five borings. These fill soils were encountered immediately below the surficial materials and extended to depths ranging from 4.5 to 12 ft below ground surface (elevations 956.0 to 974.6 ft amsl). Based on laboratory testing results and a visual review of the fill samples obtained, the embankment fills at the site are primarily classified as Silt and Clay (A-6a), and Silty Clay (A-6b). With respect to the soil strength, the soils can be described as having a relative consistency ranging from stiff to hard correlating to converted SPT-N values (N_{60}) between 7 and 26 blows per foot (bpf). Natural moisture contents of the fill ranged from 15% to 26% in moisture. Thin layers of Gravel and Stone Fragments (A-1-b and A-2-4) were also encountered as fill materials in B-027-5-17 and B-027-6-17.

The soil stratum encountered immediately beneath the embankment fill generally consisted of cohesive, fine grained glacial till soils. Based on laboratory testing results and a visual review of the samples obtained, the glacial till soils encountered are comprised primarily of Silty Clay (A-6b), and Silt and Clay(A-6a). With respect to the soil strength, the soils can be described as having a relative consistency ranging from stiff to hard correlating to converted SPT-N values (N_{60}) between 11 and 25 blows per foot (bpf). Natural moisture contents of the cohesive soils ranged from 16% to 29% in moisture.

In Boring B-027-4-16, B-027-6-17, and B-027-7-17, layers of Coarse and Fine Sand (A-3a) and Gravel and Stone Fragments with Sand (A-1-b) were encountered at a depth of 12.5 ft and 22 ft below ground surface (bgs). Additionally, Boring B-027-4-16 terminated in coarse and fine sand (A-3a). These granular materials were intermittent in the cohesive stratum described above. With respect to the soil strength, the natural non-cohesive soils encountered can be characterized as having a relative compactness of medium dense to dense, correlating to converted SPT-N values (N_{60}) between 24 and 42 blows per foot (bpf). Natural moisture contents of the granular soils ranged from 8% to 16% in moisture.

The last stratum encountered at 23.4 ft and 29.5 ft bgs is a soil layer of Silt (A-4b) and Sandy Silt (A-4a) in boring B-027-6-17, and B-027-7-17. With respect to the soil strength, the soils can be described as medium stiff to hard cohesive soils at the top of the stratum and gradually becomes medium dense as granular soils, correlating to converted SPT-N values (N_{60}) between 6 and 20 blows per foot (bpf). Natural moisture contents of the silt soils ranged from 12% to 25% in moisture. The silt soils have low plasticity with respect to the plastic index ranging from NP to 8.

4.1.2. Groundwater

Groundwater measurements were attempted during the boring drilling procedures and immediately following the completion of the borehole. Free water was encountered in five out of eight borings (see Table 3) during drilling performed as part of the referenced project. Static water elevation after completion of drilling was not recorded in any of the retaining wall borings.

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

Table 3: Groundwater Summary

Boring ID	Free Water Depth (ft)	Free Water Elevation (ft)	Static Water Depth (ft)	Static Water Elevation (ft)
Retaining Wall #1				
B-014-1-16	14.0	945.2	-	-
B-015-1-16	22.0	938.0	-	-
Retaining Wall #4				
B-027-6-17	40.0	942.0	-	-
B-027-7-17	25.0	957.4	-	-

5. ANALYSIS AND RECOMMENDATIONS

We understand that the construction of RW#1 is proposed to retain the eastbound of SR-18 from STA 126+89.28 to STA 129+05.28. Based on the site plan and cross section provided by GPD Group, Inc. (GPD), dated March 1, 2018, RW#1 will be a modular block wall. For much of the length of the wall along SR-18, the roadside face of the wall will be offset 52 ft from the centerline of construction of SR-18. The proposed wall is anticipated to be approximately 216 ft in length and will have a maximum exposed wall height of approximately 7.0 ft.

The construction of RW#4 is planned along SR-18 and River Styx Road in the southwest quadrant of the intersection of the two referenced roadways. The proposed retaining wall will be located approximately offset 52 ft from the centerline of construction of SR-18 alignment and starts from approximate STA 167+40 (proposed SR-18) with a curve that extends south along River Styx Rd to approximate STA 920+63 for a total length of about 336 ft. It is also our understanding that the proposed retaining wall is planned to have a maximum exposed height approximately 9.3 ft. According to the email from GPD Group dated January 4, 2016, the preferred wall type is a solider pile wall with temporary wood lagging and permanent concrete wall facing. Based on planned roadway grades and alignment, AASHTO's LRFD BDS dictates that the planned wall shall be designed for a live load surcharge of 240 pound per square foot (psf).

Based on the above information in addition to: 1) the soil characteristics gathered during the subsurface exploration (i.e., SPT results, laboratory test results, etc.); 2) the developed generalized soil profile at the proposed wall location and other design assumptions presented in subsequent sections of this report; and, 3) the proposed retaining wall plans provided by GPD Group on December 21, 2016 and March 1, 2018, geotechnical analyses consisting of bearing resistance for RW#1, external stability (i.e., overall stability, and maximum moment) for RW#4, global stability and seismic analysis for the project site were performed.

The geotechnical engineering analyses were performed in accordance with ODOT's BDM (ODOT, 2007) and AASHTO's LRFD BDS (AASHTO, 2014). Based on the results of the analysis, it is our opinion that the subsurface conditions encountered are generally satisfactory and will provide adequate resistance assuming the proposed RW#1 and RW#4 are constructed in accordance with the recommendations provided within this report, as well as all applicable standards and specifications (i.e., ODOT, manufacture, etc.) for soldier pile wall construction.

5.1. Generalized Soil Profile for Analysis

For analysis purposes, each boring log was reviewed and a generalized soil profile was developed for analysis. Utilizing the generalized soil profile, engineering properties for each soil strata was estimated based on their field (i.e., SPT N_{60} Values, hand penetrometer values, etc.) and laboratory (i.e., Atterberg Limits, grain size, etc.) test results using correlations provided in published engineering manuals, research reports and guidance documents. Engineering soil properties were estimated for each individual classified layer per boring location. The summary of the generalized soil profile including designated soil types, elevations, average engineering soil properties per boring location are presented in Tables 4 through 13 below.

Table 4: Soil Profile and Estimated Engineering Properties – B-015-0-14 at Retaining Wall #1

Retaining Wall #1: Soil Profile, B-015-0-14						
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)
Silty Clay Depth (959.6 ft - 950.1 ft)	115	115	125	3150	250	27
Silty Clay Depth (950.1 ft - 947.6 ft)	118	118	128	3500	300	28
Silt and Clay Depth (947.6 ft - 940.3 ft)	125	125	135	6150	400	33
Silty Clay Depth (940.3 ft - 938.6 ft)	110	110	120	1600	150	23
Sandy Silt Depth (938.6 ft - 928.1 ft)	120	110	120	1550	150	24

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 5: Soil Profile and Estimated Engineering Properties – B-016-0-14 at Retaining Wall #1

Retaining Wall #1: Soil Profile, B-016-0-14						
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)
Silt and Clay Depth (959.9 ft - 956.9 ft)	108	108	118	1100	100	22
Clay Depth (956.9 ft - 952.6 ft)	110	110	120	1550	150	22
Silty Clay Depth (952.6 ft - 948.6 ft)	110	110	120	1500	150	23
Clay Depth (948.6 ft - 944.6 ft)	110	110	120	1250	150	22
Gravel with Sand and Silt Depth (944.6 ft - 940.6 ft)	132	122	132	-	-	40
Silty Clay Depth (940.6 ft - 936.6 ft)	135	125	135	6100	400	33
Silt and Clay Depth (936.6 ft - 932.6 ft)	118	108	118	1100	100	22
Sandy Silt Depth (932.6 ft - 929.9 ft)	125	115	125	3100	250	28

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 6: Soil Profile and Estimated Engineering Properties – B-014-1-16 at Retaining Wall #1

Retaining Wall #1: Soil Profile, B-014-1-16						
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)
Silty Clay Depth (959.2 ft - 947.2 ft)	110	110	120	1550	150	23
Silt and Clay Depth (947.2 ft - 942.2 ft)	125	115	125	3400	250	27
Silty Clay Depth (942.2 ft - 937.2 ft)	120	110	120	1600	150	23
Clay Depth (937.2 ft - 935.2 ft)	120	110	120	1500	150	22
Silt and Clay Depth (935.2 ft - 929.7 ft)	122	112	122	1900	200	24
Sandy Silt Depth (929.7 ft - 924.7 ft)	128	118	128	4350	300	30
Silt and Clay Depth (924.7 ft - 919.7 ft)	135	125	135	5550	400	32
Sandy Silt Depth (919.7 ft - 912.2 ft)	135	125	135	5500	400	33
Silt Elevation (912.2 ft - 899.7 ft)	125	115	125	2800	250	27
Silt and Clay Elevation (899.7 ft - 897.7 ft)	135	125	135	5600	400	32

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 7: Soil Profile and Estimated Engineering Properties – B-015-1-16 at Retaining Wall #1

Retaining Wall #1: Soil Profile, B-015-1-16						
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)
Depth (959.2 ft - 927.5 ft)				-		
Silt and Clay Depth (927.5 ft - 915.5 ft)	130	120	130	5250	350	31
Silty Clay Depth (915.5 ft - 908 ft)	128	118	128	3850	300	28
Clay Depth (908 ft - 898.5 ft)	125	115	125	3450	250	26

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 8: Soil Profile and Estimated Engineering Properties – B-016-1-16 at Retaining Wall #1

Retaining Wall #1: Soil Profile, B-016-1-16						
Soil Description	Unit Weight(1) (pcf)	Moist Unit Weight(1) (pcf)	Saturated Unit Weight(1) (pcf)	Undrained Shear Strength(2) (psf)	Effective Cohesion(3) (psf)	Effective Friction Angle(3) (degrees)
Depth (960.9 ft - 928.4 ft)				-		
Sandy Silt Depth (928.4 ft - 925.5 ft)	115	115	125	2750	250	27
Silt and Clay Depth (925.5 ft - 909.4 ft)	120	120	130	5050	350	31

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$<52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 9: Soil Profile and Estimated Engineering Properties B-027-3-16 at Retaining Wall #4

Retaining Wall #4: Soil Profile, B-027-3-16						
Soil Description	**Unit Weight(1) (pcf)**	**Moist Unit Weight(1) (pcf)**	**Saturated Unit Weight(1) (pcf)**	**Undrained Shear Strength(2) (psf)**	**Effective Cohesion(3) (psf)**	**Effective Friction Angle(3) (degrees)**
Silt and Clay Elevation (968 ft - 961 ft)	108	108	118	1050	100	22
Silty Clay Elevation (961 ft - 958.5 ft)	112	112	122	1750	200	24
Silt and Clay Elevation (958.5 ft - 956 ft)	110	110	120	1350	150	23
Silt and Clay Elevation (956 ft - 951 ft)	112	112	122	2050	200	24
Silt Elevation (951 ft - 946.5 ft)	108	108	118	1100	100	23

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$ <52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 10: Soil Profile and Estimated Engineering Properties B-027-4-16 at Retaining Wall #4

Retaining Wall #4 : Soil Profile, B-027-4-16						
Soil Description	**Unit Weight(1) (pcf)**	**Moist Unit Weight(1) (pcf)**	**Saturated Unit Weight(1) (pcf)**	**Undrained Shear Strength(2) (psf)**	**Effective Cohesion(3) (psf)**	**Effective Friction Angle(3) (degrees)**
Silty Clay Elevation (979.1 ft - 974.6 ft)	110	110	120	1250	150	23
Silty Clay Elevation (974.6 ft - 966.6 ft)	115	115	125	2600	250	26
Coarse and Fine Sand Elevation (966.6 ft - 961.6 ft)	118	118	128	-	-	37
Silt and Clay Elevation (961.6 ft - 958.8 ft)	115	115	125	3100	250	27
Coarse and Fine Sand Elevation (958.8 ft - 957.6 ft)	120	120	130	-	-	38

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$ <52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 11: Soil Profile and Estimated Engineering Properties B-027-5-17 at Retaining Wall #4

Retaining Wall #4 : Soil Profile, B-027-5-17						
Soil Description	**Unit Weight(1) (pcf)**	**Moist Unit Weight(1) (pcf)**	**Saturated Unit Weight(1) (pcf)**	**Undrained Shear Strength(2) (psf)**	**Effective Cohesion(3) (psf)**	**Effective Friction Angle(3) (degrees)**
Gravel with Sand Elevation (974.2 ft - 971.7 ft)	112	112	122	-	-	36
Silty Clay Elevation (971.7 ft - 964.7 ft)	112	112	122	1750	200	24
Silt and Clay Elevation (964.7 ft - 959.7 ft)	115	115	125	2550	250	26
Silty Clay Elevation (959.7 ft - 949.7 ft)	112	112	122	1900	200	24
Silt and Clay Elevation (949.7 ft - 944.7 ft)	112	112	122	1800	200	24
Silt Elevation (944.7 ft - 942.2 ft)	112	112	122	-	-	30
Silt Elevation (942.2 ft - 927.7 ft)	118	118	128	3500	300	29

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$ <52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 12: Soil Profile and Estimated Engineering Properties B-027-6-17 at Retaining Wall #4

Retaining Wall #4 : Soil Profile, B-027-6-17						
Soil Description	Unit Weight$^{(1)}$ (pcf)	Moist Unit Weight$^{(1)}$ (pcf)	Saturated Unit Weight$^{(1)}$ (pcf)	Undrained Shear Strength$^{(2)}$ (psf)	Effective Cohesion$^{(3)}$ (psf)	Effective Friction Angle$^{(3)}$ (degrees)
Silt and Clay Elevation (982 ft - 973 ft)	112	112	122	2300	200	25
Gravel with Sand and Silt Elevation (973 ft - 970 ft)	115	115	125	-	-	37
Silt and Clay Elevation (970 ft - 960 ft)	112	112	122	2100	200	25
Gravel with Sand Elevation (960 ft - 958.6 ft)	115	115	125	-	-	37
Silt Elevation (958.6 ft - 943.7 ft)	115	115	125	-	-	30
Silt Elevation (943.7 ft - 935.5 ft)	120	110	120	-	-	27

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$ <52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

Table 13: Soil Profile and Estimated Engineering Properties B-027-7-17 at Retaining Wall #4

Retaining Wall #4 : Soil Profile, B-027-7-17						
Soil Description	Unit Weight$^{(1)}$ (pcf)	Moist Unit Weight$^{(1)}$ (pcf)	Saturated Unit Weight$^{(1)}$ (pcf)	Undrained Shear Strength$^{(2)}$ (psf)	Effective Cohesion$^{(3)}$ (psf)	Effective Friction Angle$^{(3)}$ (degrees)
Silt and Clay Elevation (982.4 ft - 970.4 ft)	112	112	122	2250	200	25
Silt and Clay Elevation (970.4 ft - 960.4 ft)	110	110	120	1550	150	23
Coarse and Fine Sand Elevation (960.4 ft - 957.4 ft)	115	115	125	-	-	34
Silt and Clay Elevation (957.4 ft - 952.9 ft)	120	110	120	1650	150	23
Sandy Silt Elevation (952.9 ft - 935.9 ft)	122	112	122	1750	200	25

Notes:
1. *Values interpreted from Geotechnical Bulletin 7 Table 1.*
2. *Values calculated from Terzaghi and Peck (1967) if $N1_{60}$ <52, else Stroud and Butler (1975) was used.*
3. *Values interpreted from Geotechnical Bulletin 7 Table 2.*

5.1. RW#1 Analyses

5.1.1. Retaining Wall Design Assumptions

As the proposed RW#1 is to be designed as gravity modular block type wall, it is assumed that NEAS's analyses responsibilities include: 1) bearing capacity recommendations at the proposed bearing wall elevation within the plans, and, 2) perform a review of global stability based on the plan indicated exposed wall heights and finished grades.

With respect to RW#1 specific design constraints and assumptions, the geometry of the proposed wall (i.e., exposed wall heights, existing ground elevations, proposed final grade behind/at the toe of the wall, etc.) is assumed to be consistent with that shown in the proposed Retaining Wall #1 plan provided by GPD Group via email on March 1, 2018.

5.1.2. Bearing Resistance

A shallow foundation bearing analysis was performed for RW#1 in general accordance with the LRFD Bridge Design Specifications, 7th Edition with 2015 interim revisions, Section 10.6.3.1.2a. Based on: 1) the developed generalized profile; 2) estimated engineering soil properties; and, 3) an estimated minimum embedment depth of 0.5 ft, bearing resistance analyses were performed for RW#1 under effective (drained) and total (undrained) stress conditions.

As RW#1 configuration and associated bearing elevation is anticipated to change along the alignment, bearing resistance was reviewed and broken down by possible bearing strata into separate segments along the length of the wall. Based on our review, RW#1 were broken down into two (2) separate bearing strata segments. Each segment (bearing soil) was evaluated for resistance to bearing pressure at the Strength Limit State in accordance with Section 11.10.5.4 of the AASHTO's LRFD BDS. Table 14 below summarizes the estimated bearing resistance for various block widths within each segment for RW#1. The table also summarizes segment (bearing soil) station range, estimated bearing elevation range, and the boring data used in analysis. Bearing Resistance Calculation Results can be found in Appendix C.

Table 14: Nominal and Factored Bearing Resistance

Station Range	Bearing Elevation Range (ft amsl)[1]	Boring Data Used in Calculations	Width (ft)	Drained Nominal Bearing Resistance (psf)	Drained Factored Bearing Resistance (psf)	Undrained Nominal Bearing Resistance (psf)	Undrained Factored Bearing Resistance (psf)
BEGIN to STA 128+05	894.41 - 955.75	B-016-0-14	1.0	3,165	1,582	8,029	4,015
			2.0	3,366	1,683	8,037	4,018
			3.0	3,566	1,783	8,044	4,022
STA 128+05 to End	957.08 - 961.08	B-016-0-14	1.0	2,315	1,158	5,714	2,857
			2.0	2,511	1,255	5,720	2,860
			3.0	2,706	1,353	5,725	2,862

Notes:

1. *Assumed Bearing Elevations based on RW Site Plan and embedment depth of 0.5 ft.*
2. *Resistance Factor of 0.5 Per LRFD Bridge Design Specifications Article 10.5.5.2.2-1.*

5.1.3. Global Stability

For purposes of evaluating the stability of the RW#1 site, NEAS reviewed a cross-section within the project limits that was interpreted to represent conditions that posed the greatest potential for slope instability. In general, cross-sections along the proposed wall alignment were reviewed to determine if the section would represent a combination of existing subsurface conditions and planned site grading that would be most critical to slope stability (i.e., maximum total wall height, maximum embankment height measured from toe of slope to top of wall coping, proposed cut into existing embankment slopes, weak or thick soil layer, etc.). Based on our review of the available information at the referenced locations and the associated soil properties, one cross-section, STA 127+49 in reference to SR-18 was estimated to be most "critical" and were analyzed for global stability.

For the cross-section, NEAS developed a representative cross-sectional model to use as the basis for global stability analyses. The model was developed from NEAS's interpretation of the available information which included: 1) The proposed Retaining Wall #1 plans provided by GPD Group; 2) a live load surcharge of 240 psf, accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this report. With respect to the soils' engineering properties, the provided Soil Profile and Estimated Engineering Properties presented in Section 5.1.1. of this report were used in our analyses.

The above referenced slope stability model was analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide 7.0* by Rock Science, Inc. Specifically, the Modified Bishop and simplified Janbu analysis methods were used to calculate a factor of safety (FOS) for circular slope failures. The FOS is the ratio of the resisting forces and the driving forces, with the desired safety factor being more than about 1.33 which equates to an AASHTO resistance factor less than 0.75 (per AASHTO's LRFD BDS the specified resistance factors are essentially the inverse of the FOS that should be targeted in slope stability programs). For this analysis, a resistance factor of 0.75 or lower is targeted as the slope does not contain or support a structural element.

Based on our slope stability analysis for the referenced retaining wall section, the minimum slope stability safety factor is about 3.92 (0.26 resistance factor). The results of the analyses are summarized in Table 15. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix C.

Table 15: RW#1 Glabal Stability Analysis Summary

Method	Description	Minimum Factor of Safety	Equivalent Resistance Factor	Status (OK/NG)
Circular Failure	Short Term	13.74	0.07	OK
	Long Term	3.92	0.26	OK

5.1.4. Recommendations

Temporary Excavations: It is recommended that all temporary excavations comply with the most recent Occupational Safety and Health Administration (OSHA) Excavating and Trenching Standard, Title 29 of the Code of Federal Regulation (CFR) Part 1926, Subpart P. The contractor is responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. Per Title 29 CFR Part 1926, the contractor's competent person should evaluate the soil exposed in the excavations as part of their safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. Based on the natural soils encountered at the site, it is recommended that temporary excavation slopes (exceeding a depth of 3 ft and less than 20 ft) be laid back to at least 1H:1V and these slopes should be braced or backfilled if the excavation slope will be maintained for more than a day.

Drainage Considerations: It is recommended that adequate drainage is maintained/controlled during and after construction of the retaining wall, and that roadway drainage is carefully controlled around the retaining wall location in order to prevent ponding, erosion of retained backfill soil, loss of shear strength of foundation soils due to saturation, and other drainage related issues. Also, it is recommended that internal drainage of each retaining wall be designed to provide positive drainage behind the wall and limit the buildup of hydrostatic pressure.

Furthermore, it is recommended that the barrier or curb at the roadway extend at least 25 ft beyond wall limits, and outlet to a piped collection system (i.e., collection basin/inlet) located beyond the extents of the wall. The designer should anticipate and address in design and detailing the possibility of water runoff from extreme events which will overtop the drainage swale and run down the wall face.

5.2. RW#4 Analyses

5.2.1. Retaining Wall Design Assumptions

As the proposed RW#4 is to be designed as a soldier pile lagging wall, ODOT's BDM and AASHTO's LRFD BDS dictate analysis parameters and design minimums/constraints to be used in the analysis and design process. The referenced parameters and design minimums/constraints that where significant to our analyses consist of the following:

- For permanent non-gravity cantilevered walls with discrete vertical wall elements, the simplified lateral earth pressure distributions are to conform to Figure 3.11.5.6-1 shown in AASHTO's LRFD BDS;
- If non-gravity cantilevered walls with discrete vertical wall elements are used for temporary applications, walls may be designed based on total stress methods of analysis and undrained shear strength parameters. For this case, the simplified lateral earth pressure distributions are consistent with Figure 3.11.5.6-5 presented in AASHTO's LRFD BDS;
- The width, b, of each vertical element shall be assumed to equal the width of the flange or diameter of the element for driven sections and the diameter of the concrete-filled hole for sections encased in concrete.

With respect to RW#4 specific design constraints and assumptions, the geometry of the proposed wall (i.e., exposed wall heights, existing ground elevations, proposed final grade behind/at the toe of the wall, etc.) is assumed to be consistent with that shown in the proposed plans provided by GPD Group.

The section of RW#4 along SR-18, the retained materials consist of stiff to hard Silt and Clay (A-6a) and Silty Clay (A-6b) encountered in borings B-027-3-16, B-027-4-16 and B-027-5-17. The average parameters for the retained soils are assumed as: γ' = 110 pcf, S_u = 1250 psf, c' =150 psf and φ' = 23^o (the equivalent internal friction φ_{Equiv}=33^o). The foundation soils consist of stiff to hard Silt and Clay (A-6a), stiff to hard Silty Clay (A-6b), and stiff Silt (A-4b) which were encountered in borings B-027-3-16, B-027-4-16 and B-027-5-17. The average parameters for the foundation soils are assumed as: γ' = 115 pcf, S_u = 1500 psf, c' =150 psf and φ' = 24^o (the equivalent internal friction φ_{Equiv}=34^o).

The section of RW#4 along River Styx Road, the retained materials consist of hard Silt and Clay (A-6a) and medium dense Gravel and Stone Fragments (A-2-4) encountered in borings B-027-6-17 and B-027-7-17. The average parameters for the retained soils are assumed as: γ' = 110 pcf, S_u = 2250 psf, c' =200 psf and φ' = 25^o (the equivalent internal friction φ_{Equiv}=34^o). The foundation soils consist of stiff to hard Silt and Clay (A-6a), stiff Sandy Silt (A-4a), stiff Silt (A-4b), Gravel and Stone Fragments (A-1-b) and Coarse and Fine Sand (A-3a) which were encountered in borings B-027-6-17 and B-027-7-17. The average parameters for the foundation soils are assumed as: γ' = 115 pcf, S_u = 1550 psf, c' =150 psf and φ' = 24^o (the equivalent internal friction φ_{Equiv}=34^o).

5.2.2. External Stability

Based on our estimated engineering soil properties and the retaining wall design assumptions provided in Section 5.1 of this report, an external stability analysis of the proposed RW#4 was performed. External stability was evaluated at the cross-sections of STA 168+50 and STA 924+43.

<u>LRFD Parameters</u>

The cross-sections were evaluated by using LRFD procedures for overall stability, maximum moment in steel and compressibility of concrete. These design issues have been evaluated for the proposed soldier pile walls using a MathCAD-based software solution that follows the AASHTO BDS, as described in Appendix D. Load and resistance factors applicable to the design of a soldier pile wall are presented below in Table 16 based on AASHTO LRFD Tables 3.4.1-1 and -2, and 11.5.6-1.

Table 16: Load and Resistance Factors for Soldier Pile Wall Analysis

Load Factors					
Group	Earth-Vertical	Earth-Horizontal	Live Load Surcharge-Vertical	Live Load Surcharge-Horizontal	Reference
	EV	EH	LLv	LLh	
Strength I	1	1.5	1.75	1.75	3.4.1-1/3.4.1-2
Resistance Factors					
Passive resistance				0.75	11.5.6-1

Based on the Wall4 Type Study Report provided by GPD Group, the preferred wall type is a drilled solider pile wall with temporary wood lagging and permanent concrete facing. The proposed wall is anticipated to be approximately 336 ft in length and will have a maximum exposed wall height of approximately 9.3 ft.

Input

Note that since this is a permanent wall, effective stress analysis (Figure 3.11.5.6-1 in AASHTO's LRFD BDS) parameters should be used for estimating the active pressure and passive resistance of the foundation soils (long-term). One exception would be the response to infrequent and temporary live loads. For the case (short-term), total stress analysis (Figure 3.11.5.6-5 in AASHTO's LRFD BDS) may be performed using the undrained shear strength (S_u) of the retained soils and the foundation soils to compute active pressure and passive resistance, respectively. Geotechnical design parameters of the proposed soldier pile and lagging wall for both analysis methods (Effective Stress analysis and Total Stress Analysis) are presented below in Table 17. The provided estimated soil engineering properties presented in Section 5.1.1. of this report were used in our analyses (φ_{Equiv} for Effective Stress analysis and S_u for Total Stress Analysis).

Table 17: Geotechnical Design Parameters for Soldier Pile Wall

Length of panel/wall L (ft)	Shaft diameter or pile width b (ft)	Maximum exposed wall height h (ft)	Design height H H=h+1.5*b (ft)	Embedment below design grade (ft)
8.0	2.5	9.3	13.1	16.0

Results

The capacity to demand ratios (CDRs) calculated for the most critical cross-sections with respect to overall stability and the calculated factored maximum moment are presented in Table 18 below. (Lateral earth distributions for effective stress and total stress methods, external stability, disturbing moment calculation results, steel section modulus check, design of timber lagging and permanent concrete facing can be found in Appendix D)

Results are expressed in terms of Capacity to Demand Ratios (CDR) that compare the factored Restorative moment to the factored disturbing moment. CDRs >=1 indicate a safe design. Based on our analyses, it is recommended that 30-inch diameter drilled shafts at 8 ft spacing with HP 14x73 are utilized as solider pile and the embedment depth of drilled shaft below the design grade is 16 ft. 30-inch diameter shafts are recommended in order to meet the requirement of minimum 3-in concrete cover in all direction around HP14X73.

Table 18: External Stability Analysis Summary

Section	Analysis method	Factored Disturbing Moment (ft-lb)	Factored Restorative Moment (ft-lb)	CDR for Overall stability	Factored Maximum Moment (ft-lb)	Status (OK/NG)
STA 168+50	Effective Stress	1538295.3	1629910.4	1.06	427,736	OK
	Total Stress	1975352.4	2160000	1.09	325,918	OK
STA 921+43	Effective Stress	1483257.9	1561997.5	1.05	411,625	OK
	Total Stress	1975352.4	2232000	1.13	324,587	OK

5.2.3. Global Stability

For purposes of evaluating the stability of the proposed retaining wall (RW#4) site, NEAS reviewed a cross-section within the project limits that was interpreted to represent conditions that posed the greatest potential for slope instability. In general, cross-sections along the proposed wall alignment were reviewed to determine if the section would represent a combination of existing subsurface conditions and planned site grading that would be most critical to slope stability (i.e., maximum total wall height, maximum embankment height measured from toe of slope to top of wall coping, proposed cut into existing embankment slopes, weak or thick soil layer, etc.). Based on our review of the available information at the referenced locations and the associated soil properties, one cross-section, STA 168+50 in reference to SR-18 was estimated to be most "critical" and were analyzed for global stability.

For the cross-section, NEAS developed a representative cross-sectional model to use as the basis for global stability analyses. The model was developed from NEAS's interpretation of the available information which included: 1) The proposed Retaining Wall #4 plans provided by GPD Group; 2) a live load surcharge of 240 psf, accounting for traffic induced loads; and, 3) test borings and laboratory data developed as part of this report. With respect to the soils' engineering properties, the provided Soil Profile and Estimated Engineering Properties presented in Section 5.1.1. of this report were used in our analyses.

The above referenced slope stability model was analyzed for long-term (Effective Stress) and short-term (Total Stress) slope stability utilizing the software entitled *Slide 7.0* by Rock Science, Inc. Specifically, the Modified Bishop and simplified Janbu analysis methods were used to calculate a factor of safety (FOS) for circular slope failure. The FOS is the ratio of the resisting forces and the driving forces, with the desired safety factor being more than about 1.33 which equates to an AASHTO resistance factor less than 0.75 (per AASHTO's LRFD BDS the specified resistance factors are essentially the inverse of the FOS that should be targeted in slope stability programs). For this analysis, a resistance factor of 0.75 or lower is targeted as the slope does not contain or support a structural element.

Based on our slope stability analysis for the referenced retaining wall section, the minimum slope stability safety factor is about 1.70 (0.59 resistance factor). The results of the analyses are summarized in Table 19. The graphical output of the slope stability program (cross-sectional model, calculated safety factor, and critical failure plane) is presented in Appendix D.

Table 19: RW#4 Glabal Stability Analysis Summary

Method	Description	Minimum Factor of Safety	Equivalent Resistance Factor	Status (OK/NG)
Circular Failure	Short Term	4.73	0.21	OK
	Long Term	1.70	0.59	OK

5.2.4. Recommendations

These recommendations are based on a review of existing data, field and laboratory testing results, and engineering analysis and judgment. The proposed retaining wall plans including its location and geometry was part of a conceptual design for the overall road improvement by GPD Group. If any element of the project evolves to be significantly different than is described therein, these recommendations should be reviewed by a geotechnical engineer to assess their continuing validity before they are incorporated into the design.

Geotechnical elements of the project should be designed in general accordance with the provisions of ODOT Bridge Design Manual (2007 with 2015 updates) and, as appropriate, AASHTO Bridge Design Specifications, Seventh Edition with current Interims, (LRFD BDS) using the Load and Resistance Factor (LRFD) method of design. Materials should be as specified in ODOT Construction and Materials Specifications (CMS) - 2016.

- Retaining wall should be designed using the soil description and properties provided in Section 5.1.

- Drilled shaft shall have at least 3 inches of concrete cover around the exterior of the pile.

- The wall should be provided with a drainage system extending the full height of the wall to prevent the buildup of high hydrostatic pressures. The drainage system may consist of a synthetic geo-drain installed on the concrete facing side of the lagging with the pervious (fabric) side of the drain installed to face the lagging. Base drainage should be provided with adequate outlets to permit flow of intercepted water through the wall.

- Timber lagging shall be placed from the top down as the excavation proceeds. Lagging shown above grade shall be installed and backfilled against prior to installing any permanent facing to minimize post construction deflections. Over-excavation required to place the timber lagging behind the flanges of the soldier piles shall be the minimum necessary to install the lagging. Construction of the wall should be conducted in accordance with relevant sections of the CMS. For geotechnical items these are likely to include, but not necessarily be limited to, the appropriate sub sections of:
    - Section 201 Clearing and Grubbing
    - Section 203 Roadway Excavation and Embankment
    - Section 204 Subgrade Compaction and Proof Rolling
    - Section 207 Temporary Sediment and Erosion Controls
    - Section 503 Excavations for Structures
    - Section 518 Drainage of Structures

- Foundation areas should be proof rolled to detect potentially weak soils, and any unsuitable material so identified should be removed and replaced with stable compacted embankment fill or granular fill.

- Soils in the base of all foundation excavations should be observed for suitability by a geotechnical engineer or soil technician working under the direct supervision of a geotechnical engineer.

5.3. Seismic Design Parameters

ODOT has indicated that the whole state lies within Seismic Zone 1. Based on the results of the subsurface exploration, the laboratory test data, and our review of the AASHTO Site Class Definition from Table 3.10.3.1-1 of the AASHTO LRFD Bridge Design Specifications, NEAS recommends a project site classification of D – stiff soil. Typically, SPT N-values within the upper 90 ft of the profile are between 15 bpf and 60 bpf. Seismic design parameters for the site were developed using USGS Seismic Design Maps per AASHTO Guide Specifications for LRFD Seismic Bridge Design and are presented in Table 20 below. The detailed report is presented in Appendix E. These values were interpreted for use in our slope stability analysis where seismic forces are considered.

Table 20: AASHTO Spectrum for 7% PE in 75 Years

Variable	Symbol (AASHTO 3.10)	Value
Latitude		41.136413
Longitude		-81.811627
Site Class		D
Peak Ground Acceleration	PGA	0.042g
Short Period Acceleration	S_s	0.090g
Long Period Acceleration	S_1	0.032g
Site Factor (zero period)	F_{PGA}	1.6
Site Factor (short period)	F_a	1.6
Site Factor (long period)	F_v	2.4
Zero period response seismic coefficient	$A_s = F_{PGA} * PGA$	0.067g
Short period response seismic coefficient (0.2 seconds)	$S_{DS} = F_a * S_s$	0.144g
Long period response seismic coefficient (1.0 second)	$S_{D1} = F_v * S_1$	0.077g

6. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of characterizing the subsurface conditions at the site of Retaining Wall#1 and Retaining Wall #4 for the MED-18-13.54 project. This report has been prepared for GPD Group, ODOT and their design consultants to be used solely in evaluating the soils underlying the retaining wall site and presenting geotechnical engineering recommendations specific to this project. The assessment of general site environmental conditions or the presence of pollutants in the soil, rock and groundwater of the site was beyond the scope of this geotechnical exploration. Our recommendations are based on the results of our field explorations, laboratory tests results from representative soil samples, and geotechnical engineering analyses. The results of the field explorations and laboratory tests, which form the basis of our recommendations, are presented in the appendices as noted. This report does not reflect any variations that may occur between the borings or elsewhere on the site, or variations whose nature and extent may not become evident until a later stage of construction. In the event that any changes in the nature, design or location of the proposed retaining walls (RW#1 and RW#4) are made, the conclusions and recommendations contained in this report should not be considered valid until they are reviewed, and have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to GPD Group in performing this geotechnical exploration for the MED-18-13.54 project. Please call if there are any questions, or if we can be of further service.

Respectfully Submitted,

Zhao Mankoci, Ph.D., E.I.
Geotechnical Engineer

Chunmei He, Ph.D., P.E.
Geotechnical Engineer

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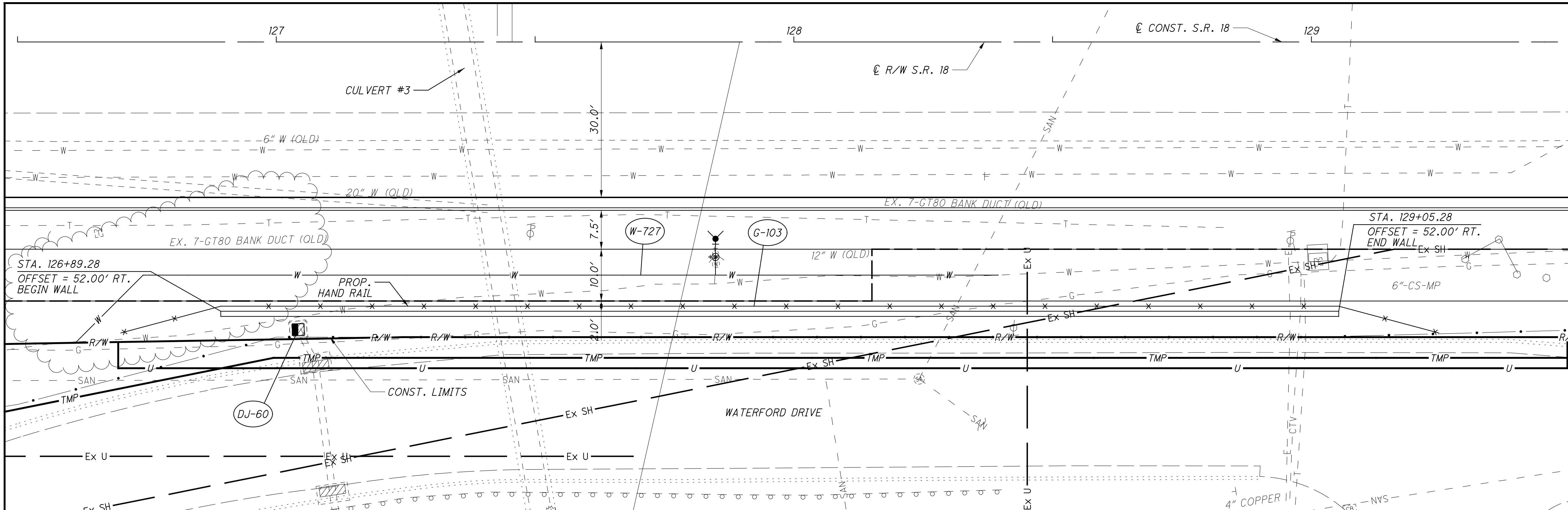
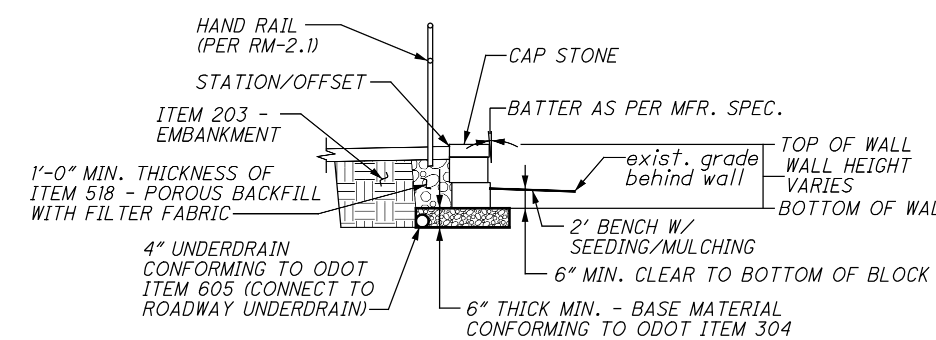
APPENDIX A

SITE PLAN



PLAN

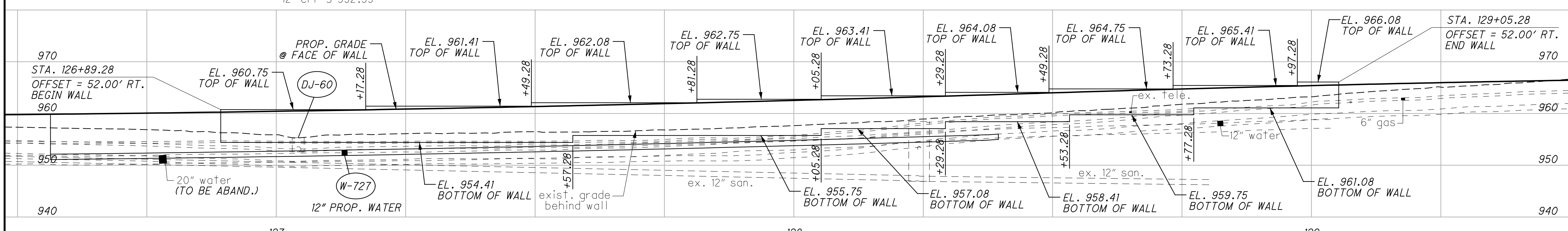
PARTICIPATION	ESTIMATED QUANTITIES - MODULAR BLOCK WALL				
	ITEM	EXT.	TOTAL	UNITS	DESCRIPTION
	512	10001		SY	SEALING OF CONCRETE SURFACES, AS PER PLAN (PERMANENT GRAFFITI PROTECTION)
	517	73501		FT	RAILING, PIPE, AS PER PLAN
	SPECIAL	61050010		SF	RETAINING WALL, MISC.: MODULAR BLOCK WALL



MODULAR BLOCK WALL

N.T.S.

DJ-60
STA 127+04.24, 55.54' RT
CB, ADJUSTED TO GRADE
PROP. GRATE ELEV 955.34
EX. GRATE ELEV 955.34
12" CPP S 952.59



WALL ELEVATION

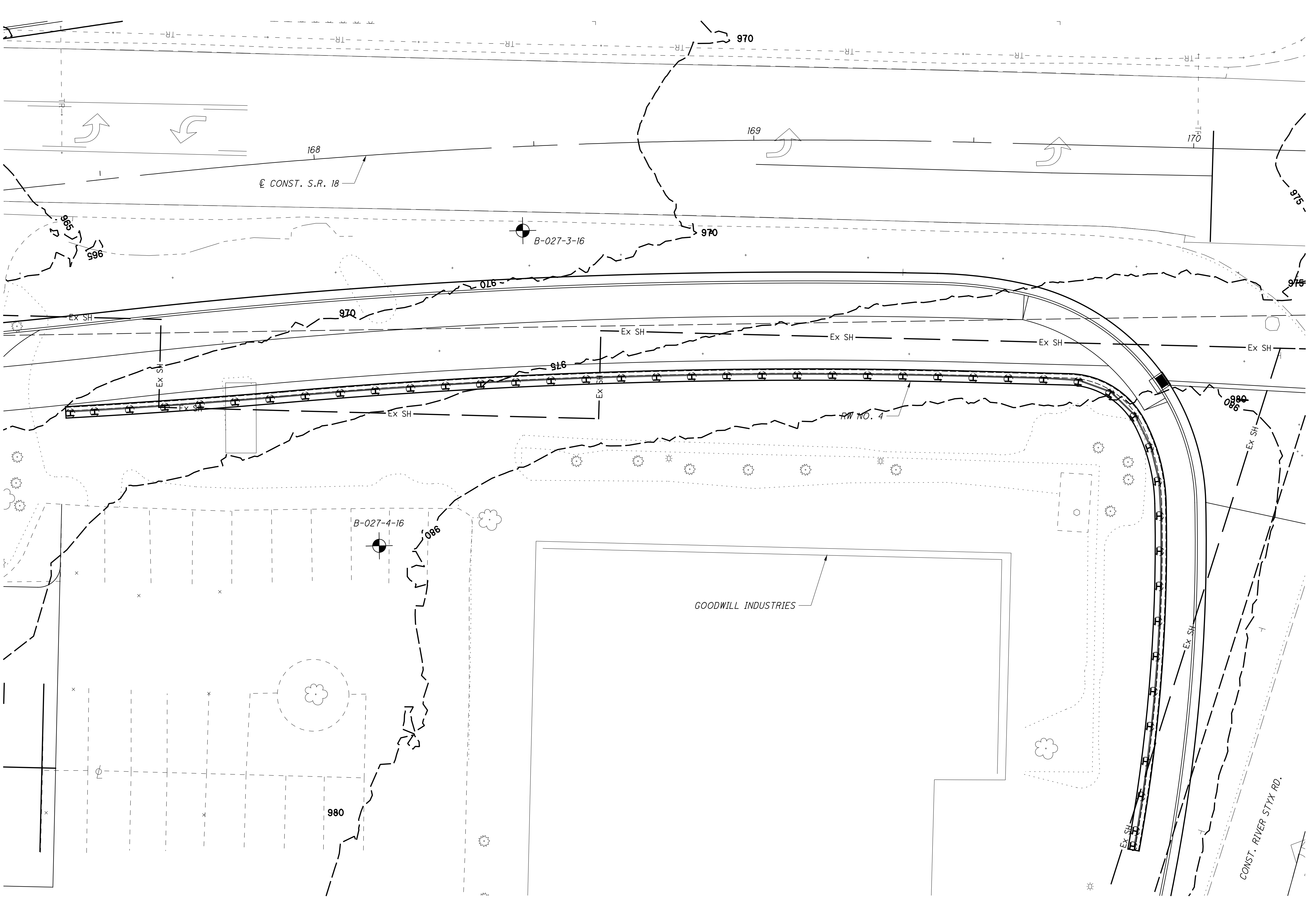
0 5 10 20
HORIZONTAL SCALE IN FEET

CALCULATED ATR
CHECKED MDG

RETAINING WALL PLAN & ELEVATION
WALL #1 - STA. 126+89.28 TO STA. 129+05.28

MED-18-12.99

O:\2013\2013013\MED-92953\STRUCTURES\92953WP001.DGN
4/26/2017
10:50:56 AM
ODOTV8ISTD_USER



HORIZONTAL SCALE IN FEET

0 5 10

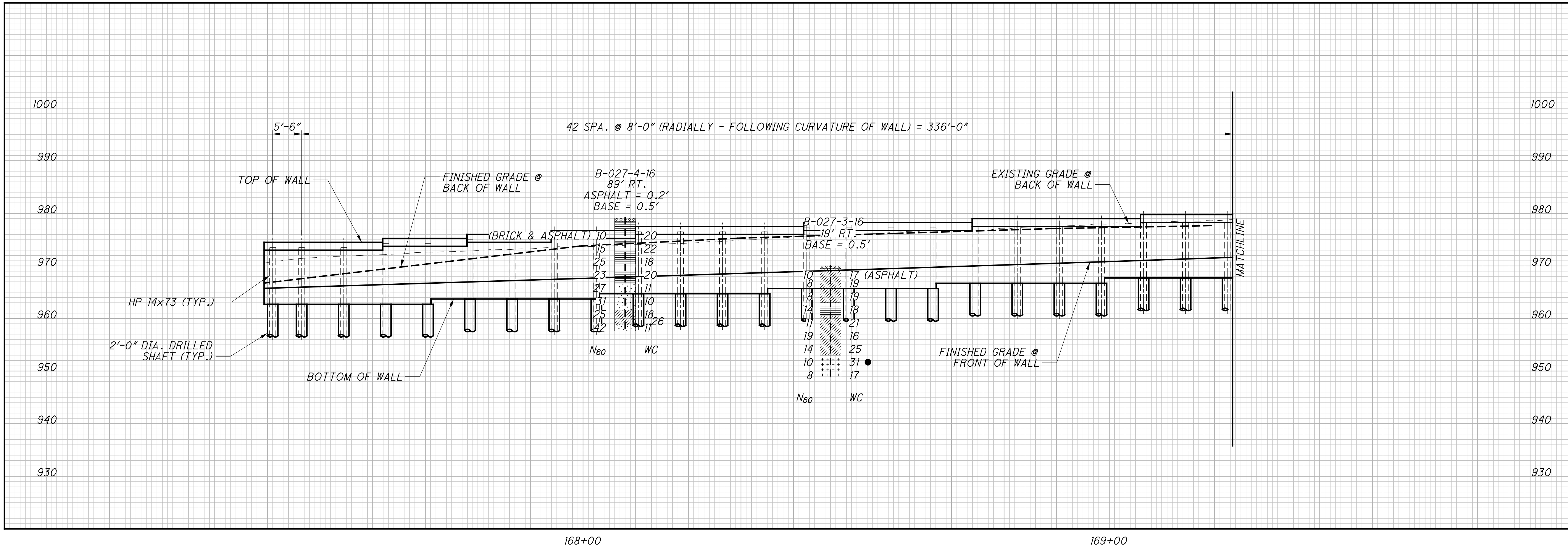
DRAWN GL

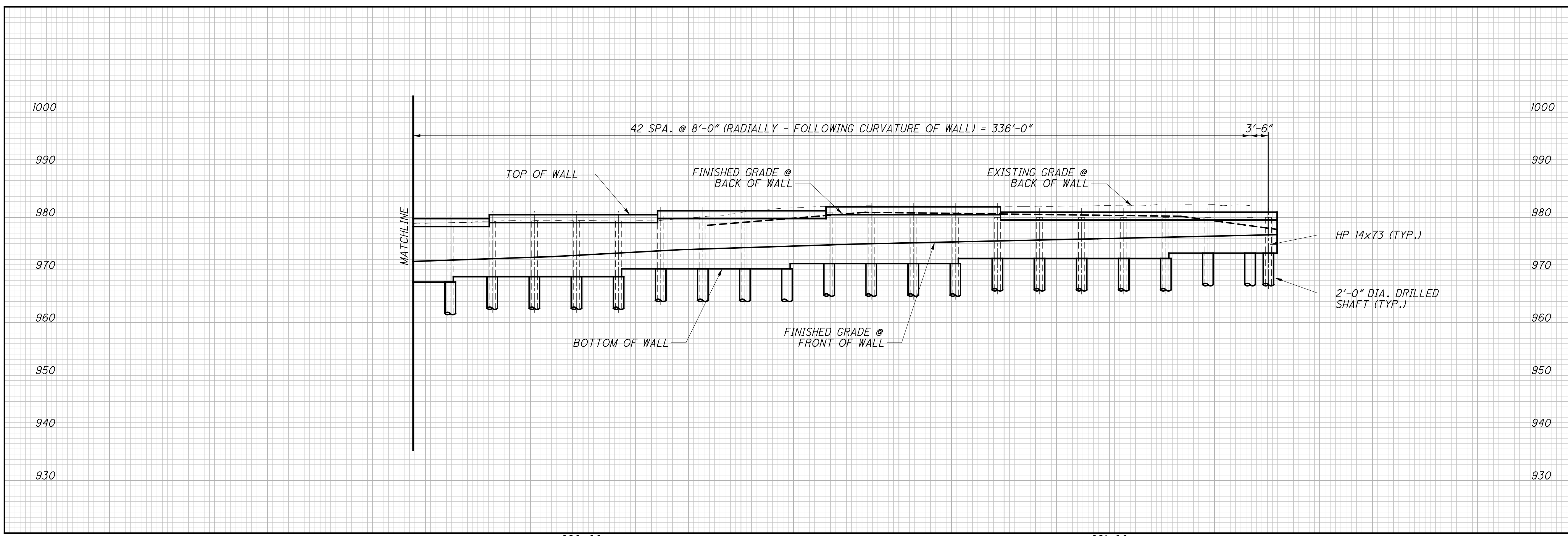
CHECKED CH

STRUCTURE FOUNDATION EXPLORATION

RETAINING WALL NO. 4

MED-18-13.54





HORIZONTAL SCALE IN FEET

DRAWN GL
CHECKED CH

STRUCTURE FOUNDATION EXPLORATION
RETAINING WALL NO. 4

MED-18-13.54

APPENDIX B

SOIL BORING LOGS

PROJECT: MED-18-12.99	DRILLING FIRM / OPERATOR: BARR / ASHBAUGH	DRILL RIG: CME 45B	STATION / OFFSET: 127+12, 23 LT
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: BARR / ASHBAUGH	HAMMER: CME AUTOMATIC	ALIGNMENT: SR-18
PID: 92953 BR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 1/26/14	ELEVATION: 959.6 (MSL), EOB: 31.5 ft.
START: 7/8/15 END: 7/8/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.4	COORD: 41.137745, -81.827227

EXPLORATION ID B-015-0-14

MATERIAL DESCRIPTION AND NOTES	ELEV. 959.6
6", **ASPHALT**	959.1
6", **BRICK**	958.6
SOFT TO VERY STIFF, BROWN CHANGING TO BROWN AND GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST **(FILL)** @7.5'; CHANGES TO VERY STIFF TO HARD	950.1
VERY STIFF TO HARD, BROWN, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP	947.6
STIFF TO HARD, SILT AND CLAY, **LITTLE SAND, TRACE GRAVEL**, DAMP @16.4'; ENCOUNTERED COBBLE	940.3
STIFF, BROWN MOTTLED WITH GRAY, **SILTY CLAY**, SOME SAND, LITTLE GRAVEL, MOIST	938.6
@22.5'; SS-10 NO RECOVERY VERY SOFT TO MEDIUM STIFF, BROWN MOTTLED WITH GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, MOIST	

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
4 7 9	21	44	SS-1	1.7 - 2.1	-	-	-	-	-	-	-	-	21	A-6b (V)
3 4 4	10	6	SS-2	0.4 - 0.5	-	-	-	-	-	-	-	-	24	A-6b (V)
4 6 7	17	39	SS-3	1.2 - 1.5	-	-	-	-	-	-	-	-	24	A-6b (V)
8 16 25	53	56	SS-4	2.0 - 4.5+	7	8	11	32	42	38	19	19	18	A-6b (11)
7 10 12	28	67	SS-5	2.0 - 4.5+	-	-	-	-	-	-	-	-	18	A-6b (V)
4 8 29	48	72	SS-6	2.0 - 4.5+	5	6	12	34	43	33	18	15	18	A-6a (10)
5 18 50/5"	-	88	SS-7	1.7 - 4.5+	-	-	-	-	-	-	-	-	16	A-6a (V)
36 50	-	83	SS-8	4.5+	-	-	-	-	-	-	-	-	17	A-6a (V)
3 5 5	13	100	SS-9	1.2 - 1.6	16	10	12	29	33	36	19	17	20	A-6b (8)
5 3 4	9	0	SS-10	-	-	-	-	-	-	-	-	-	-	
4 3 3	8	22	SS-11	0.5 - 0.8	-	-	-	-	-	-	-	-	29	A-4a (V)
WOH 3 3	8	100	SS-12	0.2 - 0.6	6	8	12	46	28	26	16	10	18	A-4a (8)

DEPTHS: 1–29; HOLE SEALED

NOTES: GROUNDWATER ENCOUNTERED AT 23.5' DURING DRILLING. CAVE DEPTH 26.0'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED .5 BAG ASPHALT PATCH; SOIL MIXED WITH BENTONITE PELLETS

PID: 92953	BR ID:	PROJECT: MED-18-12.99	STATION / OFFSET: 127+11.51, 23.0 LT	START: 7/8/15	END: 7/8/15	PG 2 OF 2	B-015-0-14

MATERIAL DESCRIPTION AND NOTES	ELEV. 929.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
@30.0'; SS-13 NO RECOVERY	928.1	31 EOB	7 9 10	25	0	SS-13	-	-	-	-	-	-	-	-	-	-		

NOTES: GROUNDWATER ENCOUNTERED AT 23.5' DURING DRILLING. CAVE DEPTH 26.0'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED .5 BAG ASPHALT PATCH; SOIL MIXED WITH BENTONITE PELLETS

PROJECT: MED-18-12.99	DRILLING FIRM / OPERATOR: BARR / J.HODGES	DRILL RIG: CME 45B	STATION / OFFSET: 127+42, 19 RT
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: BARR / C.PATRICK	HAMMER: CME AUTOMATIC	ALIGNMENT: SR-18
PID: 92953 BR ID:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 1/26/14	ELEVATION: 959.9 (MSL), EOB: 30.0 ft.
START: 7/2/15 END: 7/2/15	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.4	COORD: 41.137619, -81.827141

EXPLORATION ID B-016-0-14

MATERIAL DESCRIPTION AND NOTES	ELEV. 959.9	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
3", **ASPHALT**	959.7																	
2", **GRANULAR BASE**	959.5																	
HARD, BROWN WITH GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP **(FILL)**	956.9		3 3 4	9	56	SS-1	4.5+	-	-	-	-	-	-	-	-	18	A-6a (V)	
VERY STIFF TO HARD, BROWN WITH GRAY, **CLAY**, SOME SILT, LITTLE SAND, TRACE GRAVEL, CONTAINS FEW ROOT HAIRS, MOIST **(FILL)** @4.5'; SS-3 CONTAINS FIELD TILL FRAGMENTS	952.6		3 4 5	12	100	SS-2	2.75 - 3.0	1	3	9	28	59	47	20	27	21	A-7-6 (16)	
			3 4 6	13	100	SS-3	2.5 - 4.5+	-	-	-	-	-	-	-	-	21	A-7-6 (V)	
VERY STIFF, BROWN WITH GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP **(POSSIBLE FILL)**	948.6		4 4 5	12	100	SS-4	2.6 - 4.0	-	-	-	-	-	-	-	-	17	A-6b (V)	
STIFF TO VERY STIFF, GRAYISH BROWN MOTTLED W/ GRAY BROWN AND DARK GRAY, **CLAY**, "AND" SILT, LITTLE SAND, CONTAINS FEW FINE ROOTS, MOIST	944.6	W 15	3 3 5	10	100	SS-5	1.4 - 3.2	0	4	9	36	51	49	21	28	24	A-7-6 (17)	
DENSE, GRAYISH BROWN, **GRAVEL WITH SAND AND SILT**, LITTLE CLAY, MOIST	940.6		7 23 13	46	78	SS-6	-	-	-	-	-	-	-	-	-	24	A-2-4 (V)	
STIFF, BROWN MOTTLED WITH GRAYISH BROWN, **SILTY CLAY**, LITTLE SAND,TRACE GRAVEL, MOIST	936.6		6 11 27	49	78	SS-7	1.4 - 1.7	-	-	-	-	-	-	-	-	23	A-6b (V)	
STIFF TO VERY STIFF, OLIVE GRAY MOTTLED WITH GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST	932.6		3 3 4	9	100	SS-8	1.25 - 4.0	-	-	-	-	-	-	-	-	22	A-6a (V)	
VERY STIFF, GRAY, **SANDY SILT**, LITTLE CLAY, TRACE GRAVEL, DAMP	929.9	EOB	4 8 11	25	100	SS-9	2.75 - 3.25	8	15	16	42	19	22	16	6	13	A-4a (5)	

NOTES: GROUNDWATER ENCOUNTERED AT 15.0' DURING DRILLING. CAVE DEPTH 20.0'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED .5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 1/20/17 14:41 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\1ARCHIVE BY YEAR\2016 ARCHIVE\MED-18-13.54 PHASE 2\GINT F

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: BEI / ASHBAUGH	DRILL RIG: CME 55	STATION / OFFSET: 126+68, 27' LT.
TYPE: LANDSLIDE	SAMPLING FIRM / LOGGER: BEI / K.BAME	HAMMER: CME AUTOMATIC	ALIGNMENT: PR S.R. 18
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/3/15	ELEVATION: 959.2 (MSL) EOB: 61.5 ft.
START: 9/20/16 END: 9/21/16	SAMPLING METHOD: SPT	ENERGY RATIO (%): 81.8	LAT / LONG: 41.137912, -81.826561

EXPLORATION ID B-014-1-16

MATERIAL DESCRIPTION AND NOTES	ELEV. 959.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
6.0", **ASPHALT**	958.7																	
8.0", **BRICK**	958.0	1																
10.0", **GRANULAR BASE**	957.2	2																
VERY STIFF TO HARD, BROWN MOTTLED WITH GRAY BECOMING GRAYISH BROWN, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST		3 – 4	3 3 4	10	72	SS-1	2.5 - 3.75	8	9	10	28	45	39	18	21	19	A-6b (12)	
		5 – 6	4 4 5	12	50	SS-2	3.0 - 3.5	-	-	-	-	-	-	-	-	19	A-6b (V)	
		7 – 9	4 3 7	14	83	SS-3	4.5+	-	-	-	-	-	-	-	-	15	A-6b (V)	
@10.0'; SS-4 CONTAINS FEW ROOTS AND DECAYED ORGANICS	947.2	10 – 11	3 5 6	15	61	SS-4	4.0- 4.5+	-	-	-	-	-	-	-	-	17	A-6b (V)	
MEDIUM STIFF TO VERY STIFF, BROWN MOTTLED WITH GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST		12 – 14 (W)	3 4 7	15	78	SS-5	3.9 - 4.0	4	6	11	37	42	33	19	14	17	A-6a (10)	
@15.0'; SS-6 BECOMES GRAYISH BROWN	942.2	15 – 16	5 20 9	40	33	SS-6	0.5 - 2.75	-	-	-	-	-	-	-	-	23	A-6a (V)	
STIFF TO VERY STIFF, GRAYISH BROWN MOTTLED WITH GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST		17 – 19	4 4 7	15	56	SS-7	1.5 - 2.3	5	7	9	31	48	40	20	20	23	A-6b (12)	
	937.2	20 – 21	4 4 4	11	17	SS-8	1.50	-	-	-	-	-	-	-	-	28	A-6b (V)	
VERY STIFF, GRAYISH BROWN MOTTLED WITH BROWN, **CLAY**, "AND" SILT, TRACE SAND, TRACE GRAVEL, CONTAINS FEW ROOT HAIRS AND HAS SLIGHT ORGANIC ODOR, MOIST	935.2	22 – 23	3 4 5	12	83	SS-9	2.25	1	1	4	48	46	47	27	20	33	A-7-6 (13)	
STIFF TO VERY STIFF, GRAYISH BROWN MOTTLED WITH BROWN AND ORANGISH BROWN, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, DAMP TO MOIST @25.0'; SS-10 CONTAINS IRON STAINS		24 – 26	3 5 7	16	100	SS-10	1.75 - 2.5	-	-	-	-	-	-	-	-	25	A-6a (V)	
@27.0'; SS-11 BECOMES BROWN MOTTLED WITH GRAY	929.7	27 – 29	2 4 7	15	94	SS-11	1.75 - 2.0	3	3	4	43	47	34	21	13	25	A-6a (9)	

PID: 92953	SFN:	PROJECT: MED-18-13.54	STATION / OFFSET: 126+68, 27' LT.	START: 9/20/16	END: 9/21/16	PG 2 OF 2	B-014-1-16

MATERIAL DESCRIPTION AND NOTES	ELEV. 929.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY STIFF TO HARD, BROWNISH GRAY, **SANDY SILT**, SOME CLAY, LITTLE GRAVEL, DAMP *(continued)*		31 – 32			92	ST-12	2.5 - 4.5+	11	9	14	38	28	23	15	8	13	A-4a (6)	
	924.7	33 – 34	7 12 14	35	100	SS-13	4.5+	-	-	-	-	-	-	-	-	12	A-6a (V)	
HARD, BROWNISH GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP		35 – 36	7 13 19	44	100	SS-14	4.5+	5	7	13	41	34	26	15	11	12	A-6a (8)	
	919.7	38 – 39	8 15 18	45	94	SS-15	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
HARD, GRAYISH BROWN, **SANDY SILT**, "AND" CLAY, TRACE GRAVEL, DAMP		40 – 41	10 19 24	59	100	SS-16	4.5+	3	5	14	40	38	28	18	10	15	A-4a (8)	
		43 – 44	7 15 18	45	100	SS-17	4.5+	-	-	-	-	-	-	-	-	16	A-4a (V)	
	912.2	45 – 46	7 9 12	29	100	SS-18	4.5+	-	-	-	-	-	-	-	-	16	A-4a (V)	
MEDIUM STIFF TO HARD, GRAYISH BROWN, **SILT**, SOME TO "AND" CLAY, TRACE SAND, TRACE GRAVEL, MOIST TO WET		48 – 49	4 8 10	25	100	SS-19	3.3 - 4.2	1	1	1	50	47	31	21	10	24	A-4b (8)	
		50 – 51	8 6 8	19	100	SS-20	2.0 - 2.2	-	-	-	-	-	-	-	-	23	A-4b (V)	
		53 – 54	3 3 3	8	100	SS-21	1.6 - 2.25	0	1	3	61	35	27	20	7	26	A-4b (8)	
@55.0'; ST-22 NO RECOVERY		55 – 57			0	ST-22	-	-	-	-	-	-	-	-	-	-		
	899.7	58 – 59	5 10 18	38	100	SS-23	0.5 - 1.5	-	-	-	-	-	-	-	-	27	A-4b (V)	
HARD, GRAYISH BROWN, **SILT AND CLAY**, SOME SAND, TRACE GRAVEL, DAMP	897.7	60 – 61 EOB	9 14 19	45	100	SS-24	4.5+	-	-	-	-	-	-	-	-	14	A-6a (V)	

NOTES: GROUNDWATER ENCOUNTERED AT 14.0' DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; PUMPED 90 GAL. BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 1/20/17 14:41 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\1ARCHIVE BY YEAR\2016 ARCHIVE\MED-18-13.54 PHASE 2\GINT F

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: BEI / ASHBAUGH	DRILL RIG: CME 55	STATION / OFFSET: 127+62, 22' LEFT	EXPLORATION ID B-015-1-16
TYPE: LANDSLIDE	SAMPLING FIRM / LOGGER: BEI / K.BAME	HAMMER: CME AUTOMATIC	ALIGNMENT: PR S.R. 18	
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/3/15	ELEVATION: 960.0 (MSL) EOB: 61.5 ft.	PAGE 1 OF 2
START: 9/20/16 END: 9/20/16	SAMPLING METHOD: SPT	ENERGY RATIO (%): 81.8	LAT / LONG: 41.137871, -81.826222	

MATERIAL DESCRIPTION AND NOTES	ELEV. 960.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
AUGERED DOWN (NO SAMPLING)		1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 W 22 23 24 25 26 27 28 29																

MATERIAL DESCRIPTION AND NOTES	ELEV. 930.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
AUGERED DOWN *(continued)*	927.5	31–32																
VERY STIFF TO HARD, GRAYISH BROWN, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, DAMP		33–34	4 6 9	20	89	SS-1	2.25 - 3.25	6	9	13	41	31	24	16	8	14	A-4a (7)	
		35–36	6 12 17	40	100	SS-2	4.5+	-	-	-	-	-	-	-	-	13	A-4a (V)	
		38–39	6 13 20	45	100	SS-3	4.5+	-	-	-	-	-	-	-	-	12	A-4a (V)	
		40–41	7 19 22	56	89	SS-4	4.0 - 4.5+	-	-	-	-	-	-	-	-	21	A-4a (V)	
	915.5	43–44	8 15 21	49	94	SS-5	4.5+	-	-	-	-	-	-	-	-	14	A-4a (V)	
STIFF TO HARD, GRAYISH BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP		45–46	8 11 16	37	100	SS-6	2.5 - 4.25	4	6	12	36	42	27	16	11	15	A-6a (8)	
		48–49	7 8 12	27	100	SS-7	2.0 - 4.0	-	-	-	-	-	-	-	-	19	A-6a (V)	
@50.0'; SS-8 BECOMES BROWN, TRACE SAND (INTERBEDDED SILT AND CLAY), MOIST	908.0	50–51	8 10 11	29	100	SS-8	1.75 - 2.25	-	-	-	-	-	-	-	-	27	A-6a (V)	
MEDIUM STIFF TO HARD, GRAYISH BROWN, **SANDY SILT**, "AND" TO LITTLE CLAY, MOIST		53–54	4 4 6	14	100	SS-9	0.5 - 1.0	0	0	1	44	55	31	21	10	27	A-4a (8)	
@55.0'; SS-10 TO SS-12 BECOME TRACE GRAVEL, DAMP		55–56	7 9 10	26	100	SS-10	1.25 - 2.0	-	-	-	-	-	-	-	-	20	A-4a (V)	
		58–59	2 3 7	14	89	SS-11	4.0 - 4.25	-	-	-	-	-	-	-	-	14	A-4a (V)	
@60.0'; SS-12 CONTAINS IRON STAINS	898.5	60–61 EOB	13 17 25	57	100	SS-12	4.5+	10	14	18	38	20	20	15	5	11	A-4a (5)	

NOTES: GROUNDWATER ENCOUNTERED AT 22.0' DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PUMPED 90 GAL. BENTONITE GROUT

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 1/20/17 14:41 - X:\ACTIVE PROJECTS\ACTIVE SOIL PROJECTS\1ARCHIVE BY YEAR\2016 ARCHIVE\MED-18-13.54 PHASE 2\GINT F

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: BEI / ASHBAUGH	DRILL RIG: CME 55	STATION / OFFSET: 127+78, 18' RT.	EXPLORATION ID B-016-1-16
TYPE: CULVERT	SAMPLING FIRM / LOGGER: BEI / K.BAME	HAMMER: CME AUTOMATIC	ALIGNMENT: PR S.R. 18	
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/3/15	ELEVATION: 960.9 (MSL) EOB: 51.5 ft.	PAGE 1 OF 2
START: 9/22/16 END: 9/22/16	SAMPLING METHOD: SPT	ENERGY RATIO (%): 81.8	LAT / LONG: 41.137757, -81.826190	

MATERIAL DESCRIPTION AND NOTES	ELEV. 960.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
AUGERED DOWN (No sampling)		1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29																

MATERIAL DESCRIPTION AND NOTES	ELEV. 930.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
AUGERED DOWN *(continued)*	928.4	31–32																
VERY STIFF, GRAY TO BROWN, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, DAMP		33–34	5 6 10	22	83	SS-1	3.75 - 4.0	5	7	12	45	31	26	17	9	15	A-4a (8)	
	925.5	35																
VERY STIFF TO HARD, GRAY, **SILT AND CLAY**, SOME SAND, TRACE GRAVEL, CONTAINS SILT LENSES, DAMP		35–36	5 10 17	37	100	SS-2	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
		38–39	8 15 24	53	100	SS-3	4.5+	10	10	11	37	32	26	15	11	11	A-6a (7)	
		40–41	8 13 20	45	100	SS-4	4.5+	-	-	-	-	-	-	-	-	12	A-6a (V)	
		43–44	7 16 21	50	100	SS-5	4.5+	-	-	-	-	-	-	-	-	13	A-6a (V)	
		45–46	7 12 19	42	100	SS-6	4.5+	-	-	-	-	-	-	-	-	15	A-6a (V)	
@47.5'; SS-7 AND SS-8 BECOME LITTLE SAND		48–49	6 8 12	27	89	SS-7	3.0 - 4.5+	3	4	9	37	47	31	18	13	18	A-6a (9)	
@50.0'; SS-8 BECOMES BROWN, (INTERBEDDED SILT AND CLAY), MOIST	909.4	50–51, EOB	6 9 13	30	100	SS-8	2.75 - 3.75	-	-	-	-	-	-	-	-	25	A-6a (V)	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; PUMPED 76 GAL. BENTONITE GROUT

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: BEI / ASHBAUGH	DRILL RIG: CME 55	STATION / OFFSET: 168+47, 19' RT.
TYPE: SIDEHILL CUT SECTION	SAMPLING FIRM / LOGGER: BEI / K.BAME	HAMMER: CME AUTOMATIC	ALIGNMENT: PR S.R. 18
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/3/15	ELEVATION: 968.0 (MSL) EOB: 21.5 ft.
START: 9/15/16 END: 9/15/16	SAMPLING METHOD: SPT	ENERGY RATIO (%): 81.8	LAT / LONG: 41.136296, -81.812359

EXPLORATION ID B-027-3-16

MATERIAL DESCRIPTION AND NOTES	ELEV. 968.0
6.0", **GRANULAR BASE**	967.5
VERY STIFF TO HARD, BROWN TO DARK BROWN CHANGING TO BROWN, **SILT AND CLAY**, LITTLE SAND, TRACE TO LITTLE GRAVEL, SS-1 CONTAINS ASPHALT FRAGMENTS, DAMP **(FILL)**	961.0
HARD, BROWN, **SILTY CLAY**, TRACE SAND, TRACE GRAVEL, DAMP **(FILL)**	958.5
VERY STIFF, BROWN, **SILT AND CLAY**, SOME SAND, TRACE GRAVEL, CONTAINS A 1.5" FINE SAND SEAM AND BLACK PASTIC TRASH, MOIST **(FILL)**	956.0
HARD, BROWN BECOMING GRAYISH BROWN, **SILT AND CLAY**, LITTLE TO SOME SAND, TRACE GRAVEL, DAMP TO MOIST	951.0
STIFF, BROWN, **SILT**, SOME CLAY, TRACE SAND, WET @20.0'; SS-9 BECOMES BROWNISH GRAY, DAMP	946.5 (EOB)

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
5 3 4	10	50	SS-1	4.25 - 4.5+	-	-	-	-	-	-	-	-	17	A-6a (V)	
2 3 3	8	83	SS-2	4.5+	8	8	10	35	39	35	20	15	19	A-6a (10)	
3 3 3	8	44	SS-3	2.25 - 2.5	-	-	-	-	-	-	-	-	19	A-6a (V)	
2 4 6	14	72	SS-4	4.5+	2	3	6	41	48	37	20	17	18	A-6b (11)	
4 4 4	11	100	SS-5	2.5 - 3.25	-	-	-	-	-	-	-	-	21	A-6a (V)	
3 5 9	19	100	SS-6	4.5+	6	10	11	39	34	30	19	11	16	A-6a (8)	
3 4 6	14	94	SS-7	4.0 - 4.5+	-	-	-	-	-	-	-	-	25	A-6a (V)	
3 3 4	10	78	SS-8	1.25 - 1.75	0	0	1	75	24	31	26	5	31	A-4b (8)	
1 2 4	8	83	SS-9	1.0 - 1.5	-	-	-	-	-	-	-	-	17	A-4b (V)	

DEPTHS: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 — EOB

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: SHOVELED SOIL CUTTINGS

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: BEI / ASHBAUGH	DRILL RIG: CME 55	STATION / OFFSET: 168+08, 89' RT.	EXPLORATION ID B-027-4-16
TYPE: SIDEHILL CUT SECTION	SAMPLING FIRM / LOGGER: BEI / K.BAME	HAMMER: CME AUTOMATIC	ALIGNMENT: PR S.R. 18	
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 12/3/15	ELEVATION: 979.1 (MSL) EOB: 21.5 ft.	PAGE 1 OF 1
START: 9/22/16 END: 9/22/16	SAMPLING METHOD: SPT	ENERGY RATIO (%): 81.8	LAT / LONG: 41.136101, -81.812483	

MATERIAL DESCRIPTION AND NOTES	ELEV. 979.1	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
2.0", **ASPHALT**	978.9																	
6.0", **GRANULAR BASE**	978.4																	
VERY STIFF, BROWN MOTTLED WITH GRAY AND DARK BROWN, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, CONTAINS BRICK AND ASPHALT FRAGMENTS, DAMP **(FILL)**	974.6	1–4	3 3 4	10	50	SS-1	3.0 - 3.3	7	6	10	36	41	40	21	19	20	A-6b (12)	
VERY STIFF TO HARD, BROWN MOTTLED WITH GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST		5–6	3 4 7	15	72	SS-2	3.0 - 3.75	-	-	-	-	-	-	-	-	22	A-6b (V)	
		7–9	6 8 10	25	67	SS-3	4.5+	2	5	9	39	45	36	20	16	18	A-6b (10)	
@10.0'; SS-4 CONTAINS CALCIUM NODULES	966.6	10–12	6 7 10	23	78	SS-4	4.5+	-	-	-	-	-	-	-	-	20	A-6b (V)	
MEDIUM DENSE TO DENSE, BROWN AND DARK GRAY, **COARSE AND FINE SAND**, SOME SILT, LITTLE GRAVEL, LITTLE CLAY, SS-5 CONTAINS IRON STAINING, MOIST		13–14	7 10 10	27	94	SS-5	-	16	29	23	21	11	21	16	5	11	A-3a (0)	
@15.0'; SS-6 GRAVEL IS MOSTLY SHALE	961.6	15–17	7 12 11	31	83	SS-6	-	-	-	-	-	-	-	-	-	10	A-3a (V)	
VERY STIFF TO HARD, BROWN WITH FEW GRAY MOTTLES BECOMING BROWN, **SILT AND CLAY**, TRACE TO LITTLE SAND, SS-7 CONTAINS TRACE SHALE GRAVEL AND FINE SAND LENSES, DAMP TO MOIST	958.8	18–19	5 7 11	25	89	SS-7	4.3 - 4.5+	-	-	-	-	-	-	-	-	18	A-6a (V)	
		20	7 12 19	42	100	SS-8A	3.0 - 3.6	0	1	2	46	51	36	22	14	26	A-6a (10)	
DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE SILT, LITTLE GRAVEL, TRACE CLAY, MOIST	957.6	21 EOB				SS-8B	-	-	-	-	-	-	-	-	-	11	A-3a (V)	

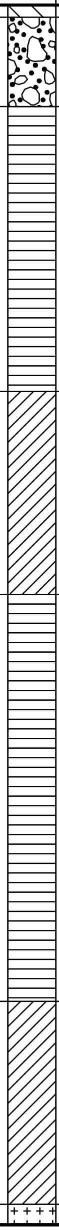
NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: PLACED 0.5 BAG ASPHALT PATCH; SHOVELED SOIL CUTTINGS

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: NEAS / J. HODGES	DRILL RIG: CME 55X	STATION / OFFSET: 167+39, 71' RT.
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: NEAS / J.HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: PR S.R. 18
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 11/21/17	ELEVATION: 974.2 (MSL) EOB: 46.5 ft.
START: 12/14/17 END: 12/14/17	SAMPLING METHOD: SPT / ST	ENERGY RATIO (%): 85.4	LAT / LONG: 41.136136, -81.812728

EXPLORATION ID B-027-5-17

MATERIAL DESCRIPTION AND NOTES	ELEV. 974.2
4.0" TOPSOIL (DRILLERS DESCRIPTION)	973.9
LOOSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, WET **(FILL)**	971.7
STIFF TO VERY STIFF, BROWN MOTTLED WITH GRAY AND ORANGISH BROWN, **SILTY CLAY**, SOME SAND, TRACE GRAVEL, CONTAINS TRACE IRON STAINING, MOIST **(FILL)**	964.7
VERY STIFF TO HARD, ORANGISH BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, CONTAINS IRON STAINING, DAMP TO MOIST	959.7
VERY STIFF, ORANGISH BROWN BECOMING GRAY, **SILTY CLAY**, TRACE SAND, TRACE GRAVEL, SS-8 CONTAINS IRON STAINING, DAMP TO MOIST	949.7
MEDIUM STIFF TO STIFF, GRAY, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, MOIST	944.7



DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
0–1.5	3 3 4	10	11	SS-1	-	-	-	-	-	-	-	-	-	21	A-1-b (V)
2.5–4	4 2 3	7	39	SS-2	1.25	-	-	-	-	-	-	-	-	26	A-6b (V)
5–6.5	3 4 7	16	89	SS-3	3.75	3	7	17	34	39	34	17	17	19	A-6b (10)
7.5–9	4 6 7	19	100	SS-4	1.25	-	-	-	-	-	-	-	-	25	A-6b (V)
10–11.5	6 7 8	21	100	SS-5	4.5+	4	6	11	34	45	35	20	15	18	A-6a (10)
12.5–14	7 7 7	20	83	SS-6	3.00	-	-	-	-	-	-	-	-	21	A-6a (V)
15–17			100	ST-7	2.75	-	-	-	-	-	-	-	-	24	A-6b (V)
17.5–19	5 7 8	21	100	SS-8	3.00	0	1	2	39	58	40	22	18	25	A-6b (11)
20–21.5	4 5 5	14	100	SS-9	2.75	-	-	-	-	-	-	-	-	18	A-6b (V)
22.5–24	3 3 5	11	100	SS-10	3.00	-	-	-	-	-	-	-	-	24	A-6b (V)
25–26.5	4 5 6	16	100	SS-11	0.75	1	2	3	36	58	35	21	14	26	A-6a (10)
27.5–29	5 4 5	13	100	SS-12	1.25	-	-	-	-	-	-	-	-	29	A-6a (V)

BACK FILL

PID: 92953 | SFN: | PROJECT: MED-18-13.54 | STATION / OFFSET: 167+39, 71' RT. | START: 12/14/17 | END: 12/14/17 | | B-027-5-17

MATERIAL DESCRIPTION AND NOTES	ELEV. 944.2	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
MEDIUM DENSE, GRAY, **SILT**, SOME CLAY, TRACE SAND, TRACE GRAVEL, WET *(continued)*	942.2	31	3 4 5	13	100	SS-13	-	0	0	1	75	24	NP	NP	NP	28	A-4b (8)	
MEDIUM STIFF, GRAY, **SILT**, "AND" CLAY, TRACE SAND, TRACE GRAVEL, MOIST		32–34	4 6 5	16	83	SS-14	1.00	1	1	1	58	39	28	20	8	25	A-4b (8)	
		35–36	6 10 12	31	100	SS-15	0.75	-	-	-	-	-	-	-	-	25	A-4b (V)	
@40.0' TO 46.5'; BECOMES VERY STIFF TO HARD, LITTLE SAND, DAMP		37–41	7 8 13	30	100	SS-16	2.50	-	-	-	-	-	-	-	-	17	A-4b (V)	
	927.7	42–46 EOB	8 10 15	36	100	SS-17	4.5+	-	-	-	-	-	-	-	-	13	A-4b (V)	

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 5 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: NEAS / J. HODGES	DRILL RIG: CME 55X	STATION / OFFSET: 169+93, 69' RT.
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: NEAS / J.HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: PR S.R. 18
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 11/21/17	ELEVATION: 982.0 (MSL) EOB: 46.5 ft.
START: 12/7/17 END: 12/7/17	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.4	LAT / LONG: 41.136148, -81.811841

EXPLORATION ID B-027-6-17

MATERIAL DESCRIPTION AND NOTES	ELEV. 982.0	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
HARD, BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE TO LITTLE GRAVEL, DAMP **(FILL)**			4 6 8	20	100	SS-1	4.5+	4	5	11	36	44	33	19	14	16	A-6a (10)	
			5 7 11	26	100	SS-2	4.5+	-	-	-	-	-	-	-	-	16	A-6a (V)	
	973.0		3 3 4	10	100	SS-3	4.5+	-	-	-	-	-	-	-	-	18	A-6a (V)	
MEDIUM DENSE, BROWN AND DARK GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, LITTLE CLAY, CONTAINS TRACE IRON STAINING AND TRACE GRANITE STONE FRAGMENTS, DAMP **(FILL)**	970.0		5 8 9	24	100	SS-4	-	36	24	12	16	12	22	14	8	10	A-2-4 (0)	
VERY STIFF TO HARD, BROWNISH GRAY, **SILT AND CLAY**, TRACE TO LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST			2 4 7	16	100	SS-5	4.00	3	6	9	38	44	33	18	15	19	A-6a (10)	
			2 5 7	17	100	SS-6	4.00	-	-	-	-	-	-	-	-	17	A-6a (V)	
			3 4 7	16	100	SS-7	4.00	-	-	-	-	-	-	-	-	17	A-6a (V)	
	960.0		3 4 9	19	100	SS-8	2.50	-	-	-	-	-	-	-	-	23	A-6a (V)	
MEDIUM DENSE, BROWN AND GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP	958.6		9 10 7	24	100	SS-9A	-	28	33	16	15	8	22	17	5	8	A-1-b (0)	
VERY STIFF TO HARD, GRAY, **SILT**, SOME CLAY, LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST						SS-9B	4.25	-	-	-	-	-	-	-	-	22	A-4b (V)	
			2 4 6	14	100	SS-10	4.25	3	9	11	51	26	23	17	6	16	A-4b (8)	
			3 5 8	19	100	SS-11	4.5+	-	-	-	-	-	-	-	-	12	A-4b (V)	

DEPTHS: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29

PID: 92953 | SFN: | PROJECT: MED-18-13.54 | STATION / OFFSET: 169+93, 69' RT. | START: 12/7/17 | END: 12/7/17 | PG 2 OF 2 | B-027-6-17

MATERIAL DESCRIPTION AND NOTES	ELEV. 952.0	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, GRAY, **SILT**, SOME CLAY, LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST *(continued)*		31	4 5 8	19	100	SS-12	4.5+	-	-	-	-	-	-	-	-	13	A-4b (V)	
		32																
		33	3 4 5	13	100	SS-13	4.5+	-	-	-	-	-	-	-	-	13	A-4b (V)	
		34																
		35–36	1 3 4	10	100	SS-14	3.75	-	-	-	-	-	-	-	-	13	A-4b (V)	
	943.7	37–38																
MEDIUM DENSE, GRAY, **SILT**, LITTLE CLAY, TRACE SAND, TRACE GRAVEL, WET		39																
		W 942.0 — 40–41	1 2 2	6	100	SS-15	-	0	2	5	79	14	NP	NP	NP	25	A-4b (8)	
		42–44																
	935.5	45–46 EOB	2 2 2	6	100	SS-16	-	-	-	-	-	-	-	-	-	19	A-4b (V)	

NOTES: GROUNDWATER ENCOUNTERED AT 40.0' DURING DRILLING. HOLE DID NOT CAVE.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 5 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

PROJECT: MED-18-13.54	DRILLING FIRM / OPERATOR: NEAS / J. HODGES	DRILL RIG: CME 55X	STATION / OFFSET: 920+68, 36' LT.	EXPLORATION ID B-027-7-17
TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: NEAS / J.HODGES	HAMMER: CME AUTOMATIC	ALIGNMENT: P.R. RIVER STYX RD	
PID: 92953 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 11/21/17	ELEVATION: 982.4 (MSL) EOB: 46.5 ft.	PAGE 1 OF 2
START: 12/6/17 END: 12/6/17	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85.4	LAT / LONG: 41.135915, -81.811858	

MATERIAL DESCRIPTION AND NOTES	ELEV. 982.4	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
HARD, BROWN AND BROWN MOTTLED WITH GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP **(FILL)**		3–4.5	2 5 8	19	100	SS-1	4.5+	4	5	10	40	41	31	18	13	15	A-6a (9)	
		5.5–7	4 7 9	23	100	SS-2	4.5+	-	-	-	-	-	-	-	-	17	A-6a (V)	
		8–9.5	3 5 7	17	100	SS-3	4.5+	-	-	-	-	-	-	-	-	17	A-6a (V)	
		10.5–12	1 3 6	13	100	SS-4	4.25	-	-	-	-	-	-	-	-	17	A-6a (V)	
VERY STIFF, BROWNISH GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP TO MOIST	970.4	13–14.5	2 3 4	10	100	SS-5	3.50	3	5	10	39	43	30	17	13	18	A-6a (9)	
		15.5–17	1 3 5	11	100	SS-6	3.75	-	-	-	-	-	-	-	-	18	A-6a (V)	
		18–19.5	2 5 7	17	100	SS-7	4.00	-	-	-	-	-	-	-	-	17	A-6a (V)	
		20.5–22	2 3 6	13	100	SS-8	3.50	-	-	-	-	-	-	-	-	17	A-6a (V)	
MEDIUM DENSE, BROWNISH GRAY, **COARSE AND FINE SAND**, SOME SILT, TRACE CLAY, TRACE GRAVEL, WET	960.4	23–24.5	3 6 11	24	100	SS-9	-	6	11	50	26	7	NP	NP	NP	16	A-3a (0)	
VERY STIFF, GRAY, **SILT AND CLAY**, TRACE SAND, TRACE GRAVEL, MOIST	957.4	W 957.4; 25.5–27	5 4 6	14	100	SS-10	3.25	0	1	2	40	57	36	22	14	23	A-6a (10)	
		28–29.5	3 5 4	13	100	SS-11	2.50	-	-	-	-	-	-	-	-	23	A-6a (V)	
	952.9																	

PID: 92953	SFN:	PROJECT: MED-18-13.54	STATION / OFFSET: 920+68, 36' LT.	START: 12/6/17	END: 12/6/17	PG 2 OF 2	B-027-7-17

MATERIAL DESCRIPTION AND NOTES	ELEV. 952.4	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
MEDIUM STIFF TO STIFF, GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, DAMP *(continued)*		31	2 3 3	9	100	SS-12	0.75	9	9	13	45	24	23	15	8	15	A-4a (7)	
		32																
		33	2 3 6	13	100	SS-13	1.00	-	-	-	-	-	-	-	-	14	A-4a (V)	
		34																
		35, 36	3 4 6	14	100	SS-14	1.75	-	-	-	-	-	-	-	-	14	A-4a (V)	
		37, 38, 39																
@40.0' TO 46.5'; BECOMES HARD		40, 41	2 7 7	20	100	SS-15	4.25	-	-	-	-	-	-	-	-	15	A-4a (V)	
		42, 43, 44																
	935.9	45, 46 EOB	2 4 6	14	100	SS-16	4.25	-	-	-	-	-	-	-	-	12	A-4a (V)	

NOTES: GROUNDWATER ENCOUNTERED AT 25.0' DURING DRILLING. CAVE-IN DEPTH 29.0'.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 5 BAGS HOLE PLUG; SHOVELED SOIL CUTTINGS

APPENDIX C

RETAINING WALL #1 ANALYSES

RETAINING WALL #1

BEARING RESISTANCE ANALYSIS

Objective: To determine the nominal bearing capacity of foundation soil for shallow foundation design.
Method: In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014, [Sect. 10.6.3.1.2].

Givens:

Footing Geometry:

$D_f := 0.5\ ft$ — Depth to base of footing below exterior grade

$L := 216\ ft$ — Assumed Footing Length

$B := 1\ ft$ — Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 22\ deg$ — Effective angle of internal friction

$\gamma_f := 110\ \frac{lbf}{ft^3}$ — Unit weight

$c'_f := 150\ \frac{lbf}{ft^2}$ — Cohesion

Undrained Conditions (Total Stress):

$\phi_f := 0\ deg$ — Angle of internal friction (Same as Drained Conditions if Sand)

$\gamma_f = 110\ \frac{lbf}{ft^3}$ — Unit weight

$c_f := 1550\ \frac{lbf}{ft^2}$ — Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$\gamma_q := 110\ \frac{lbf}{ft^3}$ — Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Groundwater Conditions:

$d_w := D_f$ — Depth of Groundwater below Ground Surface in front of Wall

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_f > 0,\ e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2,\ 1.0\right) \qquad N_q = 7.82$$

$$N_c := \text{if}\left(\phi'_f > 0,\ \frac{N_q - 1}{\tan(\phi'_f)},\ 5.14\right) \qquad N_c = 16.88$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f) \qquad N_\gamma = 7.1$$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right) \qquad s_c = 1.002$$

$$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L} \cdot \tan\left(\phi'_f\right)\right), 1\right) \qquad s_q = 1.002$$

$$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right) \qquad s_\gamma = 0.998$$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$$i_q := 1 \qquad i_q = 1$$

$$i_\gamma := 1 \qquad i_\gamma = 1$$

$$i_c := 1 \qquad i_c = 1$$

Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:

$$C_{wq} := \text{if}\left(d_w > D_f, 1.0, \frac{(1.0 - 0.5)}{(D_f - 0)} \cdot (d_w - 0) + 0.5\right) \qquad C_{wq} = 1$$

$$C_{w\gamma} := \text{if}\left(d_w < D_f, 0.5, \text{if}\left(d_w > 1.5 \cdot B + D_f, 1.0, \frac{(1.0 - 0.5)}{(1.5 \cdot B + D_f - D_f)} \cdot (d_w - D_f) + 0.5\right)\right) \qquad C_{w\gamma} = 0.5$$

Depth Correction Factor Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. It can be assumed that the soils above the footing are as competent as those beneath the footing. Therefore; the depth correction factor is taken as 1.0 if Df/B is less than 1.0.

$\frac{D_f}{B} = 0.5$ **<---- CHECK** **The depth correction factor is taken as 1.0 if Df/B is less than 1.0. Otherwise check [Table 10.6.3.1.2a-4]**. $d_q := 1.0$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$$N_{cm} := N_c \cdot s_c \cdot i_c \qquad N_{cm} = 16.919$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \qquad N_{qm} = 7.836$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \qquad N_{\gamma m} = 7.115$$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma} \qquad q_{nd} = 3164.5 \, \frac{lbf}{ft^2}$$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Rd} := \phi_b \cdot q_{nd}$ $q_{Rd} = 1.6 \, ksf$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi_f > 0, e^{\pi \cdot \tan(\phi_f)} \cdot \tan\left(45\ deg + \frac{\phi_f}{2}\right)^2, 1.0\right)$ $\qquad N_q = 1$

$N_c := \text{if}\left(\phi_f > 0, \frac{N_q - 1}{\tan(\phi_f)}, 5.14\right)$ $\qquad N_c = 5.14$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_f)$ $\qquad N_\gamma = 0$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right)$ $\qquad s_c = 1.001$

$s_q := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L} \cdot \tan(\phi_f)\right), 1\right)$ $\qquad s_q = 1$

$s_\gamma := \text{if}\left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right)$ $\qquad s_\gamma = 1$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$i_q := 1$ $\qquad i_q = 1$

$i_\gamma := 1$ $\qquad i_\gamma = 1$

$i_c := 1$ $\qquad i_c = 1$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad N_{cm} = 5.145$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad N_{qm} = 1$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad N_{\gamma m} = 0$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1:**

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad q_{nu} = 8029.4\ \frac{lbf}{ft^2}$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ $\qquad$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\qquad q_{Ru} = 4\ ksf$ $\qquad$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 1.6\ ksf$

Undrained Conditions: $q_{Ru} = 4\ ksf$

Objective: **To determine the nominal bearing capacity of foundation soil for shallow foundation design.**
Method: **In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014, [Sect. 10.6.3.1.2].**

Givens:

Footing Geometry:

$D_f := 0.5\ ft$ — Depth to base of footing below exterior grade

$L := 216\ ft$ — Assumed Footing Length

$B := 2\ ft$ — Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 22\ deg$ — Effective angle of internal friction

$\gamma_f := 110\ \frac{lbf}{ft^3}$ — Unit weight

$c'_f := 150\ \frac{lbf}{ft^2}$ — Cohesion

Undrained Conditions (Total Stress):

$\phi_f := 0\ deg$ — Angle of internal friction (Same as Drained Conditions if Sand)

$\gamma_f = 110\ \frac{lbf}{ft^3}$ — Unit weight

$c_f := 1550\ \frac{lbf}{ft^2}$ — Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$\gamma_q := 110\ \frac{lbf}{ft^3}$ — Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Groundwater Conditions:

$d_w := D_f$ — Depth of Groundwater below Ground Surface in front of Wall

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right) \qquad N_q = 7.82$$

$$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right) \qquad N_c = 16.88$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f) \qquad N_\gamma = 7.1$$

<u>Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:</u>

$$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right) \qquad s_c = 1.004$$

$$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L} \cdot \tan\left(\phi'_f\right)\right), 1\right) \qquad s_q = 1.004$$

$$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right) \qquad s_\gamma = 0.996$$

<u>Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**</u>

$$i_q := 1 \qquad i_q = 1$$

$$i_\gamma := 1 \qquad i_\gamma = 1$$

$$i_c := 1 \qquad i_c = 1$$

<u>Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:</u>

$$C_{wq} := \text{if}\left(d_w > D_f, 1.0, \frac{(1.0 - 0.5)}{(D_f - 0)} \cdot (d_w - 0) + 0.5\right) \qquad C_{wq} = 1$$

$$C_{w\gamma} := \text{if}\left(d_w < D_f, 0.5, \text{if}\left(d_w > 1.5 \cdot B + D_f, 1.0, \frac{(1.0 - 0.5)}{(1.5 \cdot B + D_f - D_f)} \cdot (d_w - D_f) + 0.5\right)\right) \qquad C_{w\gamma} = 0.5$$

Depth Correction Factor Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. It can be assumed that the soils above the footing are as competent as those beneath the footing. Therefore; the depth correction factor is taken as 1.0 if Df/B is less than 1.0.

$$\frac{D_f}{B} = 0.3$$ **<---- CHECK** **The depth correction factor is taken as 1.0 if Df/B is less than 1.0. Otherwise check [Table 10.6.3.1.2a-4]**. $d_q := 1.0$

<u>Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**</u>

$$N_{cm} := N_c \cdot s_c \cdot i_c \qquad N_{cm} = 16.955$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \qquad N_{qm} = 7.85$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \qquad N_{\gamma m} = 7.102$$

<u>Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**</u>

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma} \qquad q_{nd} = 3365.6 \ \frac{lbf}{ft^2}$$

<u>Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**</u>

$\phi_b := 0.5$ — Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\qquad q_{Rd} = 1.7\ ksf$ — Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi_f > 0, e^{\pi \cdot \tan(\phi_f)} \cdot \tan\left(45\ deg + \frac{\phi_f}{2}\right)^2, 1.0\right)$ $\quad N_q = 1$

$N_c := \text{if}\left(\phi_f > 0, \frac{N_q - 1}{\tan(\phi_f)}, 5.14\right)$ $\quad N_c = 5.14$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_f)$ $\quad N_\gamma = 0$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right)$ $\quad s_c = 1.002$

$s_q := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L} \cdot \tan(\phi_f)\right), 1\right)$ $\quad s_q = 1$

$s_\gamma := \text{if}\left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right)$ $\quad s_\gamma = 1$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$i_q := 1$ $\quad i_q = 1$

$i_\gamma := 1$ $\quad i_\gamma = 1$

$i_c := 1$ $\quad i_c = 1$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 5.15$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\quad N_{qm} = 1$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 0$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1:**

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nu} = 8036.8\ \frac{lbf}{ft^2}$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ $\quad$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\quad q_{Ru} = 4\ ksf$ $\quad$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 1.7\ ksf$

Undrained Conditions: $q_{Ru} = 4\ ksf$

Objective: **To determine the nominal bearing capacity of foundation soil for shallow foundation design.**
Method: **In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014, [Sect. 10.6.3.1.2].**

Givens:

Footing Geometry:

$D_f := 0.5\ ft$ Depth to base of footing below exterior grade

$L := 216\ ft$ Assumed Footing Length

$B := 3\ ft$ Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 22\ deg$ Effective angle of internal friction

$\gamma_f := 110\ \frac{lbf}{ft^3}$ Unit weight

$c'_f := 150\ \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

$\phi_f := 0\ deg$ Angle of internal friction (Same as Drained Conditions if Sand)

$\gamma_f = 110\ \frac{lbf}{ft^3}$ Unit weight

$c_f := 1550\ \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$\gamma_q := 110\ \frac{lbf}{ft^3}$ Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Groundwater Conditions:

$d_w := D_f$ Depth of Groundwater below Ground Surface in front of Wall

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right) \qquad N_q = 7.82$$

$$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right) \qquad N_c = 16.88$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f) \qquad N_\gamma = 7.1$$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right) \qquad s_c = 1.006$$

$$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L} \cdot \tan\left(\phi'_f\right)\right), 1\right) \qquad s_q = 1.006$$

$$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right) \qquad s_\gamma = 0.994$$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$$i_q := 1 \qquad i_q = 1$$

$$i_\gamma := 1 \qquad i_\gamma = 1$$

$$i_c := 1 \qquad i_c = 1$$

Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:

$$C_{wq} := \text{if}\left(d_w > D_f, 1.0, \frac{(1.0 - 0.5)}{(D_f - 0)} \cdot (d_w - 0) + 0.5\right) \qquad C_{wq} = 1$$

$$C_{w\gamma} := \text{if}\left(d_w < D_f, 0.5, \text{if}\left(d_w > 1.5 \cdot B + D_f, 1.0, \frac{(1.0 - 0.5)}{(1.5 \cdot B + D_f - D_f)} \cdot (d_w - D_f) + 0.5\right)\right) \qquad C_{w\gamma} = 0.5$$

Depth Correction Factor Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. It can be assumed that the soils above the footing are as competent as those beneath the footing. Therefore; the depth correction factor is taken as 1.0 if Df/B is less than 1.0.

$$\frac{D_f}{B} = 0.2$$ **<---- CHECK** **The depth correction factor is taken as 1.0 if Df/B is less than 1.0. Otherwise check [Table 10.6.3.1.2a-4]**.

$$d_q := 1.0$$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$$N_{cm} := N_c \cdot s_c \cdot i_c \qquad N_{cm} = 16.991$$

$$N_{qm} := N_q \cdot s_q \cdot i_q \qquad N_{qm} = 7.865$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma \qquad N_{\gamma m} = 7.088$$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**

$$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma} \qquad q_{nd} = 3566.1 \ \frac{lbf}{ft^2}$$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$$\phi_b := 0.5$$

Bearing resistance factor **LRFD 10.5.5.2.2-1**

$$q_{Rd} := \phi_b \cdot q_{nd} \qquad q_{Rd} = 1.8 \ ksf$$

Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi_f > 0, e^{\pi \cdot \tan(\phi_f)} \cdot \tan\left(45\ deg + \frac{\phi_f}{2}\right)^2, 1.0\right)$ $\qquad N_q = 1$

$N_c := \text{if}\left(\phi_f > 0, \frac{N_q - 1}{\tan(\phi_f)}, 5.14\right)$ $\qquad N_c = 5.14$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_f)$ $\qquad N_\gamma = 0$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right)$ $\qquad s_c = 1.003$

$s_q := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L} \cdot \tan(\phi_f)\right), 1\right)$ $\qquad s_q = 1$

$s_\gamma := \text{if}\left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right)$ $\qquad s_\gamma = 1$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$i_q := 1$ $\qquad i_q = 1$

$i_\gamma := 1$ $\qquad i_\gamma = 1$

$i_c := 1$ $\qquad i_c = 1$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\qquad N_{cm} = 5.154$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\qquad N_{qm} = 1$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\qquad N_{\gamma m} = 0$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1:**

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\qquad q_{nu} = 8044.1\ \frac{lbf}{ft^2}$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ $\qquad$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\qquad q_{Ru} = 4\ ksf$ $\qquad$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 1.8\ ksf$

Undrained Conditions: $q_{Ru} = 4\ ksf$

Objective: **To determine the nominal bearing capacity of foundation soil for shallow foundation design.**
Method: **In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014, [Sect. 10.6.3.1.2].**

Givens:

Footing Geometry:

$D_f := 0.5\ ft$ — Depth to base of footing below exterior grade

$L := 216\ ft$ — Assumed Footing Length

$B := 1\ ft$ — Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 22\ deg$ — Effective angle of internal friction

$\gamma_f := 108\ \frac{lbf}{ft^3}$ — Unit weight

$c'_f := 100\ \frac{lbf}{ft^2}$ — Cohesion

Undrained Conditions (Total Stress):

$\phi_f := 0\ deg$ — Angle of internal friction (Same as Drained Conditions if Sand)

$\gamma_f = 108\ \frac{lbf}{ft^3}$ — Unit weight

$c_f := 1100\ \frac{lbf}{ft^2}$ — Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$\gamma_q := 110\ \frac{lbf}{ft^3}$ — Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Groundwater Conditions:

$d_w := D_f$ — Depth of Groundwater below Ground Surface in front of Wall

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right) \qquad N_q = 7.82$$

$$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right) \qquad N_c = 16.88$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f) \qquad N_\gamma = 7.1$$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\right)\cdot\left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5\cdot L}\right)\right) \qquad s_c = 1.002$$

$$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\cdot\tan\left(\phi'_f\right)\right), 1\right) \qquad s_q = 1.002$$

$$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4\cdot\left(\frac{B}{L}\right), 1\right) \qquad s_\gamma = 0.998$$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$$i_q := 1 \qquad i_q = 1$$

$$i_\gamma := 1 \qquad i_\gamma = 1$$

$$i_c := 1 \qquad i_c = 1$$

Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:

$$C_{wq} := \text{if}\left(d_w > D_f, 1.0, \frac{(1.0-0.5)}{(D_f - 0)}\cdot\left(d_w - 0\right) + 0.5\right) \qquad C_{wq} = 1$$

$$C_{w\gamma} := \text{if}\left(d_w < D_f, 0.5, \text{if}\left(d_w > 1.5\cdot B + D_f, 1.0, \frac{(1.0-0.5)}{(1.5\cdot B + D_f - D_f)}\cdot\left(d_w - D_f\right) + 0.5\right)\right) \qquad C_{w\gamma} = 0.5$$

Depth Correction Factor Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. It can be assumed that the soils above the footing are as competent as those beneath the footing. Therefore; the depth correction factor is taken as 1.0 if Df/B is less than 1.0.

$\frac{D_f}{B} = 0.5$ **<---- CHECK** **The depth correction factor is taken as 1.0 if Df/B is less than 1.0. Otherwise check [Table 10.6.3.1.2a-4]**. $d_q := 1.0$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$$N_{cm} := N_c\cdot s_c\cdot i_c \qquad N_{cm} = 16.919$$

$$N_{qm} := N_q\cdot s_q\cdot i_q \qquad N_{qm} = 7.836$$

$$N_{\gamma m} := N_\gamma\cdot s_\gamma\cdot i_\gamma \qquad N_{\gamma m} = 7.115$$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**

$$q_{nd} := c'_f\cdot N_{cm} + \gamma_q\cdot D_f\cdot N_{qm}\cdot d_q\cdot C_{wq} + 0.5\cdot\gamma_f\cdot B\cdot N_{\gamma m}\cdot C_{w\gamma} \qquad q_{nd} = 2315\ \frac{lbf}{ft^2}$$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Rd} := \phi_b\cdot q_{nd}$ $q_{Rd} = 1.2\ ksf$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi_f > 0, e^{\pi \cdot \tan(\phi_f)} \cdot \tan\left(45\ deg + \frac{\phi_f}{2}\right)^2, 1.0\right)$ $\quad N_q = 1$

$N_c := \text{if}\left(\phi_f > 0, \frac{N_q - 1}{\tan(\phi_f)}, 5.14\right)$ $\quad N_c = 5.14$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_f)$ $\quad N_\gamma = 0$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right)$ $\quad s_c = 1.001$

$s_q := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L} \cdot \tan(\phi_f)\right), 1\right)$ $\quad s_q = 1$

$s_\gamma := \text{if}\left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right)$ $\quad s_\gamma = 1$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$i_q := 1$ $\quad i_q = 1$

$i_\gamma := 1$ $\quad i_\gamma = 1$

$i_c := 1$ $\quad i_c = 1$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 5.145$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\quad N_{qm} = 1$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 0$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1:**

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nu} = 5714.2\ \frac{lbf}{ft^2}$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ — Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\quad q_{Ru} = 2.9\ ksf$ — Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 1.2\ ksf$

Undrained Conditions: $q_{Ru} = 2.9\ ksf$

Objective: **To determine the nominal bearing capacity of foundation soil for shallow foundation design.**
Method: **In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014, [Sect. 10.6.3.1.2].**

Givens:

Footing Geometry:

$D_f := 0.5\ ft$ Depth to base of footing below exterior grade

$L := 216\ ft$ Assumed Footing Length

$B := 2\ ft$ Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 22\ deg$ Effective angle of internal friction

$\gamma_f := 108\ \frac{lbf}{ft^3}$ Unit weight

$c'_f := 100\ \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

$\phi_f := 0\ deg$ Angle of internal friction (Same as Drained Conditions if Sand)

$\gamma_f = 108\ \frac{lbf}{ft^3}$ Unit weight

$c_f := 1100\ \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$\gamma_q := 110\ \frac{lbf}{ft^3}$ Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Groundwater Conditions:

$d_w := D_f$ Depth of Groundwater below Ground Surface in front of Wall

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right) \qquad N_q = 7.82$$

$$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right) \qquad N_c = 16.88$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f) \qquad N_\gamma = 7.1$$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right)$ $\quad s_c = 1.004$

$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L} \cdot \tan\left(\phi'_f\right)\right), 1\right)$ $\quad s_q = 1.004$

$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right)$ $\quad s_\gamma = 0.996$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$i_q := 1$ $\quad i_q = 1$

$i_\gamma := 1$ $\quad i_\gamma = 1$

$i_c := 1$ $\quad i_c = 1$

Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:

$C_{wq} := \text{if}\left(d_w > D_f, 1.0, \frac{(1.0 - 0.5)}{(D_f - 0)} \cdot (d_w - 0) + 0.5\right)$ $\quad C_{wq} = 1$

$C_{w\gamma} := \text{if}\left(d_w < D_f, 0.5, \text{if}\left(d_w > 1.5 \cdot B + D_f, 1.0, \frac{(1.0 - 0.5)}{(1.5 \cdot B + D_f - D_f)} \cdot (d_w - D_f) + 0.5\right)\right)$ $\quad C_{w\gamma} = 0.5$

Depth Correction Factor Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. It can be assumed that the soils above the footing are as competent as those beneath the footing. Therefore; the depth correction factor is taken as 1.0 if Df/B is less than 1.0.

$\frac{D_f}{B} = 0.3$ **<---- CHECK** **The depth correction factor is taken as 1.0 if Df/B is less than 1.0. Otherwise check [Table 10.6.3.1.2a-4]**. $\quad d_q := 1.0$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 16.955$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\quad N_{qm} = 7.85$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 7.102$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**

$q_{nd} := c'_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nd} = 2510.8\ \frac{lbf}{ft^2}$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ $\quad$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Rd} := \phi_b \cdot q_{nd}$ $\quad q_{Rd} = 1.3\ ksf$ $\quad$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi_f > 0, e^{\pi \cdot \tan(\phi_f)} \cdot \tan\left(45\ deg + \frac{\phi_f}{2}\right)^2, 1.0\right)$ $\quad N_q = 1$

$N_c := \text{if}\left(\phi_f > 0, \frac{N_q - 1}{\tan(\phi_f)}, 5.14\right)$ $\quad N_c = 5.14$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_f)$ $\quad N_\gamma = 0$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right)$ $\quad s_c = 1.002$

$s_q := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L} \cdot \tan(\phi_f)\right), 1\right)$ $\quad s_q = 1$

$s_\gamma := \text{if}\left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right)$ $\quad s_\gamma = 1$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$i_q := 1$ $\quad i_q = 1$

$i_\gamma := 1$ $\quad i_\gamma = 1$

$i_c := 1$ $\quad i_c = 1$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 5.15$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\quad N_{qm} = 1$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 0$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1:**

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nu} = 5719.5\ \frac{lbf}{ft^2}$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ $\quad$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\quad q_{Ru} = 2.9\ ksf$ $\quad$ Factored bearing resistance Undrained Conditions

Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 1.3\ ksf$

Undrained Conditions: $q_{Ru} = 2.9\ ksf$

Objective: **To determine the nominal bearing capacity of foundation soil for shallow foundation design.**
Method: **In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014, [Sect. 10.6.3.1.2].**

Givens:

Footing Geometry:

$D_f := 0.5\ ft$ Depth to base of footing below exterior grade

$L := 216\ ft$ Assumed Footing Length

$B := 3\ ft$ Width/Breadth of footing

Foundation Soil Design Parameters:

Drained Conditions (Effective Stress):

$\phi'_f := 22\ deg$ Effective angle of internal friction

$\gamma_f := 108\ \frac{lbf}{ft^3}$ Unit weight

$c'_f := 100\ \frac{lbf}{ft^2}$ Cohesion

Undrained Conditions (Total Stress):

$\phi_f := 0\ deg$ Angle of internal friction (Same as Drained Conditions if Sand)

$\gamma_f = 108\ \frac{lbf}{ft^3}$ Unit weight

$c_f := 1100\ \frac{lbf}{ft^2}$ Cohesion (Use Su if Angle of internal friction = 0 deg)

Foundation Surcharge Soil Parameters:

$\gamma_q := 110\ \frac{lbf}{ft^3}$ Unit weight of Soil above bearing depth (Used in Bearing Resistance of Soil Calculation LRFD 10.6.3.1.2a-1)

Groundwater Conditions:

$d_w := D_f$ Depth of Groundwater below Ground Surface in front of Wall

Drained Conditions (Effective Stress):

$$N_q := \text{if}\left(\phi'_f > 0, e^{\pi \cdot \tan(\phi'_f)} \cdot \tan\left(45\ deg + \frac{\phi'_f}{2}\right)^2, 1.0\right) \qquad N_q = 7.82$$

$$N_c := \text{if}\left(\phi'_f > 0, \frac{N_q - 1}{\tan(\phi'_f)}, 5.14\right) \qquad N_c = 16.88$$

$$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi'_f) \qquad N_\gamma = 7.1$$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$$s_c := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\right)\cdot\left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5\cdot L}\right)\right) \qquad s_c = 1.006$$

$$s_q := \text{if}\left(\phi'_f > 0, 1 + \left(\frac{B}{L}\cdot\tan\left(\phi'_f\right)\right), 1\right) \qquad s_q = 1.006$$

$$s_\gamma := \text{if}\left(\phi'_f > 0, 1 - 0.4\cdot\left(\frac{B}{L}\right), 1\right) \qquad s_\gamma = 0.994$$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$$i_q := 1 \qquad i_q = 1$$

$$i_\gamma := 1 \qquad i_\gamma = 1$$

$$i_c := 1 \qquad i_c = 1$$

Compute groundwater depth correction factors per **LRFD [Table 10.6.3.1.2a-2]**:

$$C_{wq} := \text{if}\left(d_w > D_f, 1.0, \frac{(1.0-0.5)}{(D_f-0)}\cdot\left(d_w-0\right)+0.5\right) \qquad C_{wq} = 1$$

$$C_{w\gamma} := \text{if}\left(d_w < D_f, 0.5, \text{if}\left(d_w > 1.5\cdot B + D_f, 1.0, \frac{(1.0-0.5)}{(1.5\cdot B + D_f - D_f)}\cdot\left(d_w - D_f\right)+0.5\right)\right) \qquad C_{w\gamma} = 0.5$$

Depth Correction Factor Compute depth correction factor per **LRFD [Table 10.6.3.1.2a-4]**. It can be assumed that the soils above the footing are as competent as those beneath the footing. Therefore; the depth correction factor is taken as 1.0 if Df/B is less than 1.0.

$$\frac{D_f}{B} = 0.2$$ **<---- CHECK** **The depth correction factor is taken as 1.0 if Df/B is less than 1.0. Otherwise check [Table 10.6.3.1.2a-4]**.

$$d_q := 1.0$$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$$N_{cm} := N_c\cdot s_c\cdot i_c \qquad N_{cm} = 16.991$$

$$N_{qm} := N_q\cdot s_q\cdot i_q \qquad N_{qm} = 7.865$$

$$N_{\gamma m} := N_\gamma\cdot s_\gamma\cdot i_\gamma \qquad N_{\gamma m} = 7.088$$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1]:**

$$q_{nd} := c'_f\cdot N_{cm} + \gamma_q\cdot D_f\cdot N_{qm}\cdot d_q\cdot C_{wq} + 0.5\cdot\gamma_f\cdot B\cdot N_{\gamma m}\cdot C_{w\gamma} \qquad q_{nd} = 2705.9\ \frac{lbf}{ft^2}$$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$$\phi_b := 0.5$$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$$q_{Rd} := \phi_b\cdot q_{nd} \qquad q_{Rd} = 1.4\ ksf$$ Factored bearing resistance Drained Conditions

Undrained Conditions (Effective Stress):

$N_q := \text{if}\left(\phi_f > 0, e^{\pi \cdot \tan(\phi_f)} \cdot \tan\left(45\ deg + \frac{\phi_f}{2}\right)^2, 1.0\right)$ $\quad N_q = 1$

$N_c := \text{if}\left(\phi_f > 0, \frac{N_q - 1}{\tan(\phi_f)}, 5.14\right)$ $\quad N_c = 5.14$

$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi_f)$ $\quad N_\gamma = 0$

Compute shape correction factors per **LRFD [Table 10.6.3.1.2a-3]**:

$s_c := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L}\right) \cdot \left(\frac{N_q}{N_c}\right), 1 + \left(\frac{B}{5 \cdot L}\right)\right)$ $\quad s_c = 1.003$

$s_q := \text{if}\left(\phi_f > 0, 1 + \left(\frac{B}{L} \cdot \tan(\phi_f)\right), 1\right)$ $\quad s_q = 1$

$s_\gamma := \text{if}\left(\phi_f > 0, 1 - 0.4 \cdot \left(\frac{B}{L}\right), 1\right)$ $\quad s_\gamma = 1$

Load inclination factors using **LRFD [10.6.3.1.2a-5] thru [10.6.3.1.2a-9]:**

$i_q := 1$ $\quad i_q = 1$

$i_\gamma := 1$ $\quad i_\gamma = 1$

$i_c := 1$ $\quad i_c = 1$

Compute modified bearing capacity factors **LRFD [Equation 10.6.3.1.2a-2 to 10.6.3.1.2a-4]:**

$N_{cm} := N_c \cdot s_c \cdot i_c$ $\quad N_{cm} = 5.154$

$N_{qm} := N_q \cdot s_q \cdot i_q$ $\quad N_{qm} = 1$

$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$ $\quad N_{\gamma m} = 0$

Compute nominal bearing resistance, **LRFD [Eq 10.6.3.1.2a-1:**

$q_{nu} := c_f \cdot N_{cm} + \gamma_q \cdot D_f \cdot N_{qm} \cdot d_q \cdot C_{wq} + 0.5 \cdot \gamma_f \cdot B \cdot N_{\gamma m} \cdot C_{w\gamma}$ $\quad q_{nu} = 5724.7\ \frac{lbf}{ft^2}$

Compute factored bearing resistance, **LRFD [Eq 10.6.3.1.1]:**

$\phi_b := 0.5$ $\quad$ Bearing resistance factor **LRFD 10.5.5.2.2-1**

$q_{Ru} := \phi_b \cdot q_{nu}$ $\quad q_{Ru} = 2.9\ ksf$ $\quad$ Factored bearing resistance Undrained Conditions

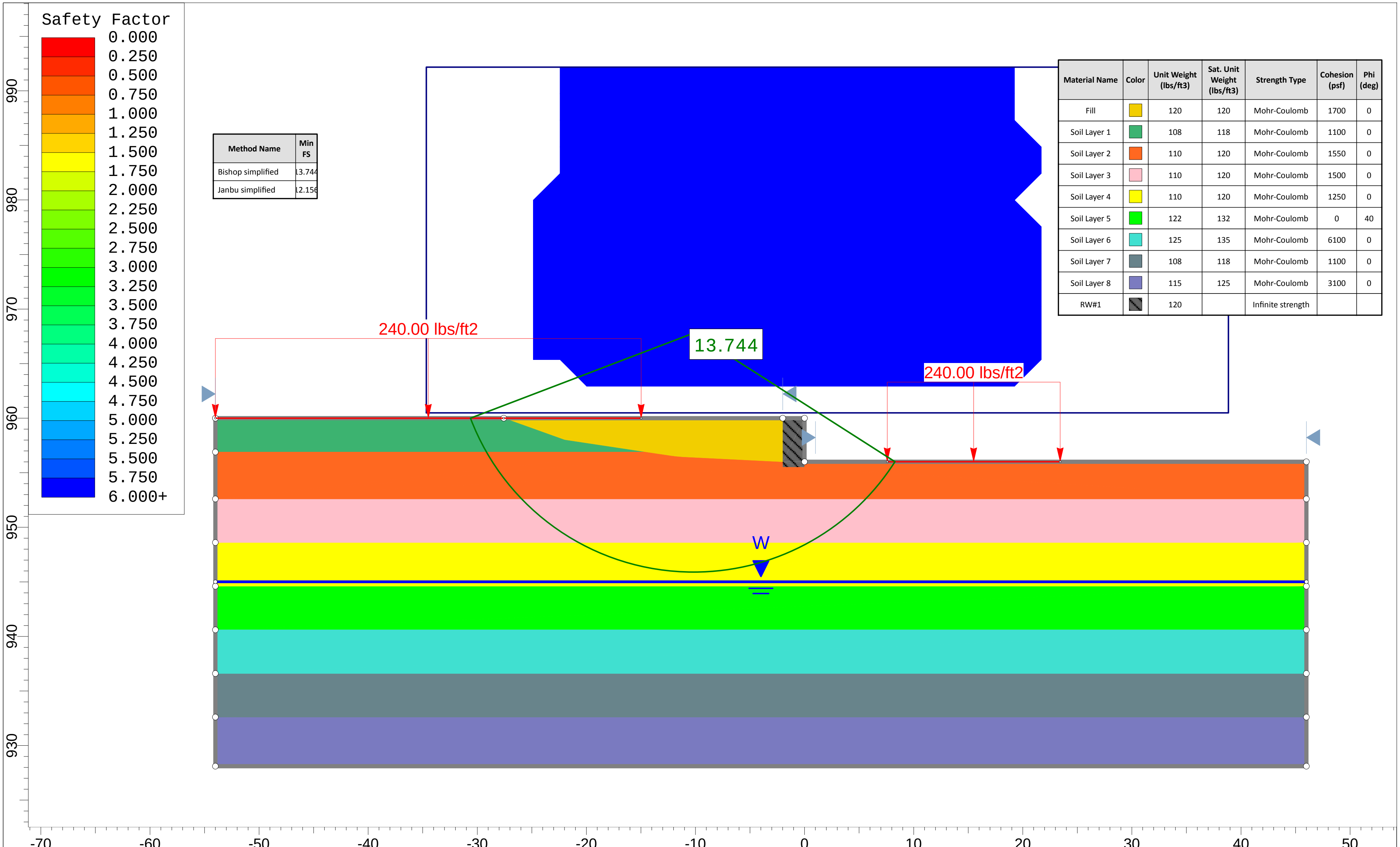
Factored Bearing Resistance Drained vs. Undrained Conditions:

Drained Conditions: $q_{Rd} = 1.4\ ksf$

Undrained Conditions: $q_{Ru} = 2.9\ ksf$

RETAINING WALL #1

GLOBAL STABILITY ANALYSIS

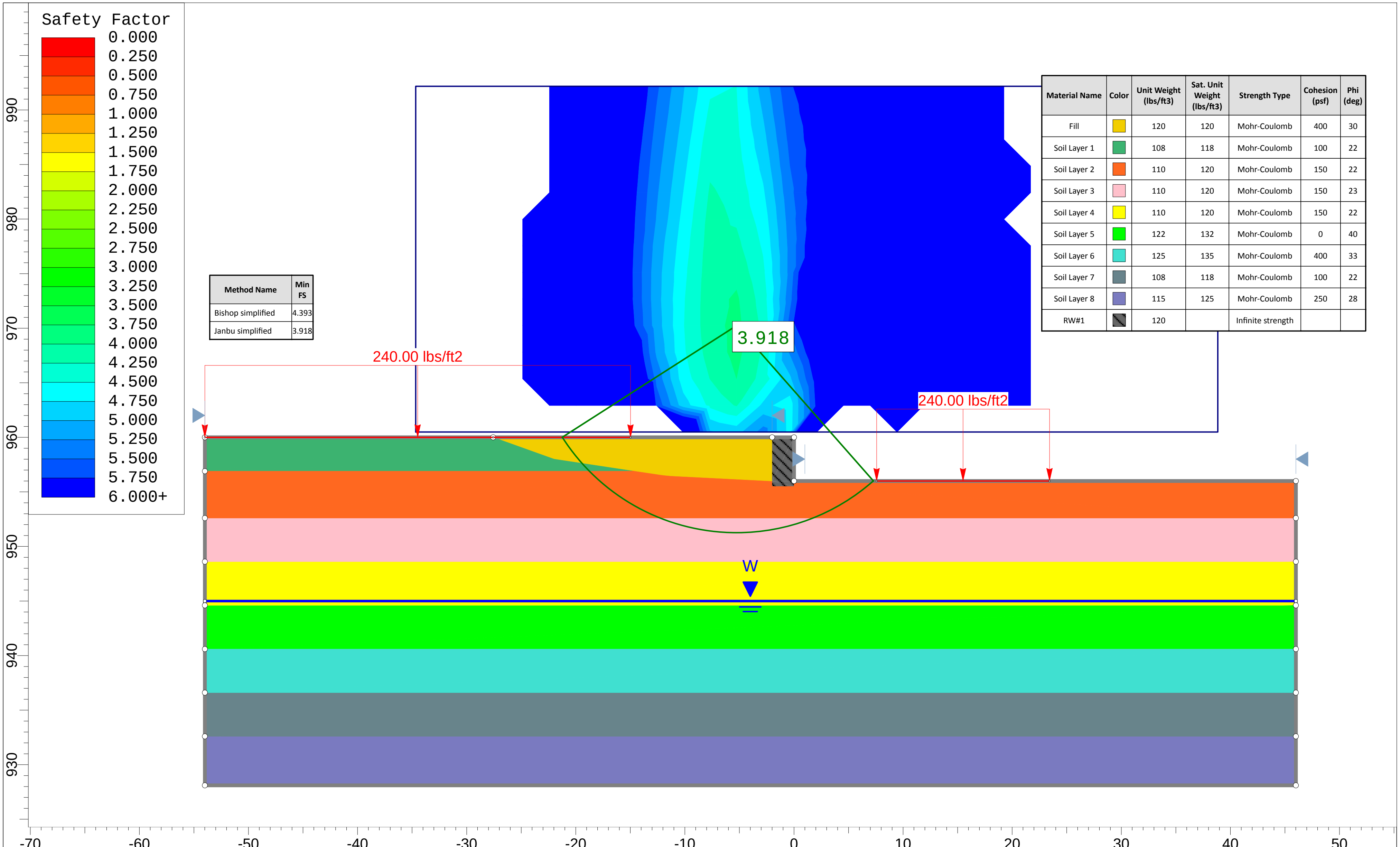


Method Name	Min FS
Bishop simplified	13.744
Janbu simplified	12.156

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		120	120	Mohr-Coulomb	1700	0
Soil Layer 1		108	118	Mohr-Coulomb	1100	0
Soil Layer 2		110	120	Mohr-Coulomb	1550	0
Soil Layer 3		110	120	Mohr-Coulomb	1500	0
Soil Layer 4		110	120	Mohr-Coulomb	1250	0
Soil Layer 5		122	132	Mohr-Coulomb	0	40
Soil Layer 6		125	135	Mohr-Coulomb	6100	0
Soil Layer 7		108	118	Mohr-Coulomb	1100	0
Soil Layer 8		115	125	Mohr-Coulomb	3100	0
RW#1		120		Infinite strength		

Project	MED-18-12.99		
Analysis Description	RW#1		
Drawn By ZM	*Scale* 1:146	*Company*	NEAS, Inc
Date	3/1/2018, 4:13:29 PM	*File Name*	RW1-STA127+81-Total Stress.slim

SLIDEINTERPRET 7.031



Method Name	Min FS
Bishop simplified	4.393
Janbu simplified	3.918

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		120	120	Mohr-Coulomb	400	30
Soil Layer 1		108	118	Mohr-Coulomb	100	22
Soil Layer 2		110	120	Mohr-Coulomb	150	22
Soil Layer 3		110	120	Mohr-Coulomb	150	23
Soil Layer 4		110	120	Mohr-Coulomb	150	22
Soil Layer 5		122	132	Mohr-Coulomb	0	40
Soil Layer 6		125	135	Mohr-Coulomb	400	33
Soil Layer 7		108	118	Mohr-Coulomb	100	22
Soil Layer 8		115	125	Mohr-Coulomb	250	28
RW#1		120		Infinite strength		

Project	MED-18-12.99			
Analysis Description	RW#1			
Drawn By	ZM	*Scale* 1:146	*Company*	NEAS, Inc
Date	3/1/2018, 4:13:29 PM		*File Name*	RW1-STA127+81-Effective Stress.slim

SLIDEINTERPRET 7.031

APPENDIX D

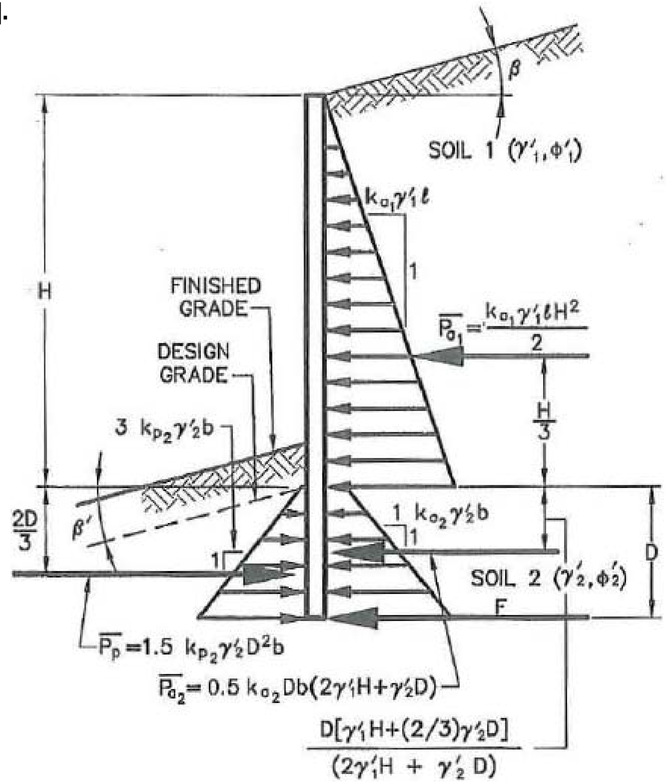
RETAINING WALL #4 ANALYSES

RETAINING WALL #4

EXTERNAL STABILITY ANALYSIS

Objective: To evaluate the stability for Soldier Pile wall with temporary wood lagging and permanent concrete facing design.
Method: In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014 and FHWA-IF-99-015
Assumptions:

- **Soldier Pile wall is treated as nongravity cantilevered wall with discrete vertical wall elements.**
- **The height of the wall to design grade H=h+1.5 b, h is height of wall and b is shaft diameter or pile width.**
- **Load and Resistance Factors Soldier Pile wall analysis are in accordance with LRFD Bridge Design Specifications, [Sect. 3.4.1-1/3.4.1-2 and 11.5.6-1].**
- **Diagram of lateral earth pressure for nongravity cantilevered wall is assumed in accordance with LRFD Bridge Design Specifications[Figure 3.11.5.6-1].**



Load and Resistance factors:

$\gamma_{EH} := 1.5$ Load factor for horizontal earth loads (LRFD [Sect. 3.4.1-1/3.4.1-2])

$\gamma_{LS} := 1.75$ Load factor for live load surcharge (LRFD [Sect. 3.4.1-1/3.4.1-2])

$\varphi_P := 0.75$ Resistance factor for passive resistance (LRFD [Sect. 11.5.7])

$\varphi_S := 1$ Resistance factor for steel flexure (LRFD [Sect. 6.5.4.2])

$\varphi_C := 0.7$ Axial Resistance factor for compressive concrete (LRFD [Sect. 5.5.4.2])

$\varphi_V := 0.9$ Resistance factor for shear and torsion (LRFD [Sect. 5.5.4.2])

Givens:

$LS := 240\ psf$ Live load surcharge

Wall Geometry:

$L := 8\ ft$ Pile spacing

$h := 9.3\ ft$ Height of wall

$b := 2.5\ ft$ Shaft diameter or pile width

$H := h + 1.5\ b$ $H = 13.1\ ft$ Design height

$D := 16\ ft$ Embedment below design grade

Soil Properties:

Retained Soil:

$\varphi_1 := 33\ deg$		Angle of internal friction
$c_1 := 0\ psf$		Cohesion
$\gamma_1 := 110\ pcf$		Unit weight
$Ka_1 := \frac{(1-\sin(\varphi_1))}{(1+\sin(\varphi_1))}$	$Ka_1 = 0.3$	Active earth pressure coefficient

Foundation Soil:

$\varphi_2 := 34\ deg$		Angle of internal friction
$c_2 := 0\ psf$		Cohesion
$\gamma_2 := 120\ pcf$		Unit weight
$Ka_2 := \frac{(1-\sin(\varphi_2))}{(1+\sin(\varphi_2))}$	$Ka_2 = 0.3$	Active earth pressure coefficient
$Kp_2 := \frac{1}{Ka_2}$	$Kp_2 = 3.5$	Passive earth pressure coefficient

Loads and Resistance:

$Pa_1 := Ka_1 \cdot \gamma_1 \cdot L \cdot \frac{H^2}{2}$	$Pa_1 = 22090.3\ lbf$	Active force-retained soil
$PaLS_1 := Ka_1 \cdot LS \cdot L \cdot H$	$PaLS_1 = 7386.5\ lbf$	Active force-live load surcharge on exposed height
$Pa_2 := \frac{1}{2} Ka_2 \cdot D \cdot b \cdot (2\ \gamma_1 \cdot H + \gamma_2 \cdot D)$	$Pa_2 = 27089.7\ lbf$	Active force-foundation soil
$PaLS_2 := Ka_2 \cdot LS \cdot b \cdot D$	$PaLS_2 = 2714.1\ lbf$	Active force-live load surcharge on embedment depth
$Pp := \frac{3}{2} Kp_2 \cdot \gamma_2 \cdot D^2 \cdot b$	$Pp = 407477.6\ lbf$	Passive resistance

Moment arms from bottom:

$La_1 := D + \frac{H}{3}$	$La_1 = 20.4\ ft$	Moment arm for active force-retained soil
$LaLS_1 := D + \frac{H}{2}$	$LaLS_1 = 22.5\ ft$	Moment arm for active force-live load on exposed height
$La_2 := D - D\ \frac{\gamma_1 \cdot H + \frac{2}{3}\ \gamma_2 \cdot D}{2\ \gamma_1 \cdot H + \gamma_2 \cdot D}$	$La_2 = 6.9\ ft$	Moment arm for active force-foundation soil
$LaLS_2 := \frac{D}{2}$	$LaLS_2 = 8\ ft$	Moment arm for active force-live load on embedment depth
$LaPp := \frac{D}{3}$	$LaPp = 5.3\ ft$	Moment arm for Passive resistance

Stability Check:

Disturbing moments:

$DM_1 := Pa_1 \cdot La_1$	$DM_1 = 449538.5\ lbf \cdot ft$	Disturbing Moment of active force-retained soil
$DMLS_1 := PaLS_1 \cdot LaLS_1$	$DMLS_1 = 166381.6\ lbf \cdot ft$	Disturbing Moment of active force-live load acting on exposed height
$DM_2 := Pa_2 \cdot La_2$	$DM_2 = 187767.9\ lbf \cdot ft$	Disturbing Moment of foundation soil
$DMLS_2 := PaLS_2 \cdot LaLS_2$	$DMLS_2 = 21712.5\ lbf \cdot ft$	Disturbing Moment of active force-live load acting on embedment depth
$M := DM_1 + DM_2$	$M = 637306.5\ lbf \cdot ft$	Total disturbing moments (without live load)
$M\gamma := M \cdot \gamma_{EH} + \left(DMLS_1 + DMLS_1\right) \cdot \gamma_{LS}$	$M\gamma = 1538295.3\ lbf \cdot ft$	Factored total disturbing moments

Restorative moments:

$RM := Pp \cdot LaPp$	$RM = 2173213.9\ lbf \cdot ft$	Restorative moment
$RM\varphi := RM \cdot \varphi_P$	$RM\varphi = 1629910.4\ lbf \cdot ft$	Factored Restorative moment

Stability Check:

$STA := \text{if}\left(RM\varphi > M\gamma, 1, 0\right)$	$STA = 1$	Stability check, "1" indicates OK, "0" indicates NG

Steel Section Modulus Check:

Determine the depth Z at which the shear in the wall is zero (i.e., The point at which the areas of the driving and resisting pressure diagrams are equivalent.) The maximum bending moment is at the point of zero shear.

To find the point of zero shear (Z):

$$Pa_2@Z = \frac{1}{2}\ Ka_2 \cdot Z \cdot b \cdot \left(2\ \gamma_1 \cdot H + \gamma_2 \cdot Z\right)$$

$$La_2@Z = Z - Z \cdot \frac{\gamma_1 \cdot H + \frac{2}{3}\ \gamma_2 \cdot Z}{2\ \gamma_1 \cdot H + \gamma_2 \cdot Z}$$

$$La_1@Z = Z + \frac{H}{3}$$

$$PaLS_2@Z = Ka_2 \cdot LS \cdot b \cdot Z$$

$$LaLS_2@Z = \frac{Z}{2}$$

$$LaLS_1@Z = Z + \frac{H}{2}$$

$$Pp@Z = \frac{3}{2}\ Kp_2 \cdot \gamma_2 \cdot Z^2 \cdot b$$

$$LaPp@Z = \frac{Z}{3}$$

$$Pa_1 + PaLS_1 + Pa_2@Z + PaLS_2@Z - Pp@Z = 0$$

Depth of zero shear below Design Grade

$$Pa_1 + PaLS_1 + \frac{1}{2}\ Ka_2 \cdot Z \cdot b \cdot \left(2\ \gamma_1 \cdot H + \gamma_2 \cdot Z\right) + Ka_2 \cdot LS \cdot b \cdot Z - \frac{3}{2}\ Kp_2 \cdot \gamma_2 \cdot Z^2 \cdot b = 0$$

$$\left(\frac{1}{2}\ Ka_2 \cdot b \cdot \gamma_2 - \frac{3}{2}\ Kp_2 \cdot \gamma_2 \cdot b\right) \cdot Z^2 + \left(Ka_2 \cdot b \cdot \gamma_1 \cdot H + Ka_2 \cdot LS \cdot b\right) \cdot Z + Pa_1 + PaLS_1 = 0$$

$$A := \left(\frac{1}{2}\ Ka_2 \cdot b \cdot \gamma_2 - \frac{3}{2}\ Kp_2 \cdot \gamma_2 \cdot b\right)$$

$$B := \left(Ka_2 \cdot b \cdot \gamma_1 \cdot H + Ka_2 \cdot LS \cdot b\right) = 38101.2\ \frac{lb}{s^2}$$

$$C := Pa_1 + PaLS_1$$

$$Z := \max\left(\frac{-B + \sqrt{B^2 - 4 \cdot A \cdot C}}{2 \cdot A}, \frac{-B - \sqrt{B^2 - 4 \cdot A \cdot C}}{2 \cdot A}\right) = 4.8\ ft$$

Depth of zero shear below Design Grade

To find the maximum moment which occurs at the point of zero shear Z:

$$MaxM = \gamma_{EH} \cdot \left(Pa_1 \cdot La_1@Z + Pa_2@Z \cdot La_2@Z\right) + \gamma_{LS} \cdot \left(PaLS_1 \cdot LaLS_1@Z + PaLS_2@Z \cdot LaLS_2@Z\right) - \varphi_P \cdot Pp@Z \cdot LaPp@Z$$

$$DM@Z := \gamma_{EH} \cdot \left(Pa_1 \cdot \left(Z + \frac{H}{3}\right) + \left(\frac{1}{2} Ka_2 \cdot Z \cdot b \cdot \left(2\,\gamma_1 \cdot H + \gamma_2 \cdot Z\right)\right) \cdot \left(Z - Z \cdot \frac{\gamma_1 \cdot H + \frac{2}{3}\gamma_2 \cdot Z}{2\,\gamma_1 \cdot H + \gamma_2 \cdot Z}\right)\right) + \gamma_{LS} \cdot \left(PaLS_1 \cdot \left(Z + \frac{H}{2}\right) + \left(Ka_2 \cdot LS \cdot b \cdot Z\right) \cdot \frac{Z}{2}\right)$$

$$RM@Z := \varphi_P \cdot \left(\frac{3}{2} Kp_2 \cdot \gamma_2 \cdot Z^2 \cdot b\right) \cdot \frac{Z}{3}$$

$MaxM := DM@Z - RM@Z$ | $MaxM = 427735.9\ \mathbf{lbf \cdot ft}$ | Maximum moment in steel

$fy := 50\ \mathbf{ksi}$ | Steel yield strength 50 ksi

$Sxr := \frac{MaxM}{\varphi_S\, fy}$ | $Sxr = 102.7\ \mathbf{in}^3$ | Request min. section modulus for steel

$Sx := 107\ \mathbf{in}^3$ | Section Modulus

$SEC := \text{if}\left(Sx > Sxr, 1, 0\right)$ | $SEC = 1$ | Steel Section Modulus check, "1" indicates OK, "0" indicates NG

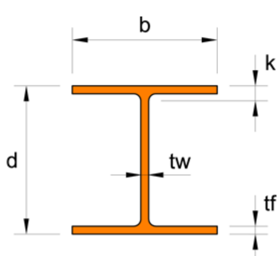
Design of timber lagging:

$S_{CTC} := L = 8\ \mathbf{ft}$ | Center to center spacing of soldier beams

$H = 13.1\ \mathbf{ft}$ | Design wall height

$t_{Lagging} := 100\ \mathbf{mm}$ | $t_{Lagging} = 3.9\ \mathbf{in}$ | Timber lagging thickness is recommened based on Table 12 in FHWA-IF-99-015

Design of permanent facing:



$fc := 4000\ \mathbf{psi}$ | Concrete compressive strength

$b := 14.6\ \mathbf{in}$ | Flange width of HP pile

$d := 13.6\ \mathbf{in}$ | Depth of HP pile

$k := 1.19\ \mathbf{in}$

$t := \frac{b}{2} - k = 6.1\ \mathbf{in}$ | Width of beam to support the lagging

$D_{Check} := \text{if}\left(D > \sqrt{b^2 + d^2} + 6\ \mathbf{in}, 1, 0\right)$ | $D_{Check} = 1$ | Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG

$\sigma_a := Ka_1 \cdot \gamma_1 \cdot H$ | $\sigma_a = 2.9\ \mathbf{psi}$ | Active pressure at bottom of facing- retained soil

$\sigma := \sigma_a \cdot \gamma_{EH} + Ka_1 \cdot LS \cdot \gamma_{LS}$ | $\sigma = 5.3\ \mathbf{psi}$ | Factored lateral pressure at bottome of facing Hf

$Ps := 1\ \mathbf{ft}$ | 1' high stip pannel

$V := \sigma \cdot L \cdot Ps$ | $V = 6068.8\ \mathbf{lbf}$ | Factored active force

$Pr := \varphi_C \cdot fc \cdot Ps \cdot t$ | $Pr = 205296\ \mathbf{lbf}$ | Resistance force on the facing

$MmaxF := \frac{1}{10} \cdot \sigma \cdot L^2 \cdot Ps$ $MmaxF = 4855\ lbf \cdot ft$ Factored max. moment at bottom of facing

$Sxc := \frac{MmaxF}{\varphi_V \cdot fc}$ $Sxc = 16.2\ in^3$ Request section modulus of concrete

$tm := \sqrt[2]{Sxc \cdot \frac{6}{Ps}}$ $tm = 2.8\ in$ Required thickness of facing based on moment

$ComC := \text{if}\left(Pr > V \wedge t > tm, 1, 0\right)$ $ComC = 1$ Compressibility of concrete check, "1" indicates OK, "0" indicates No

Design results:

$STA = 1$ Stability check, "1" indicates OK, "0" indicates No

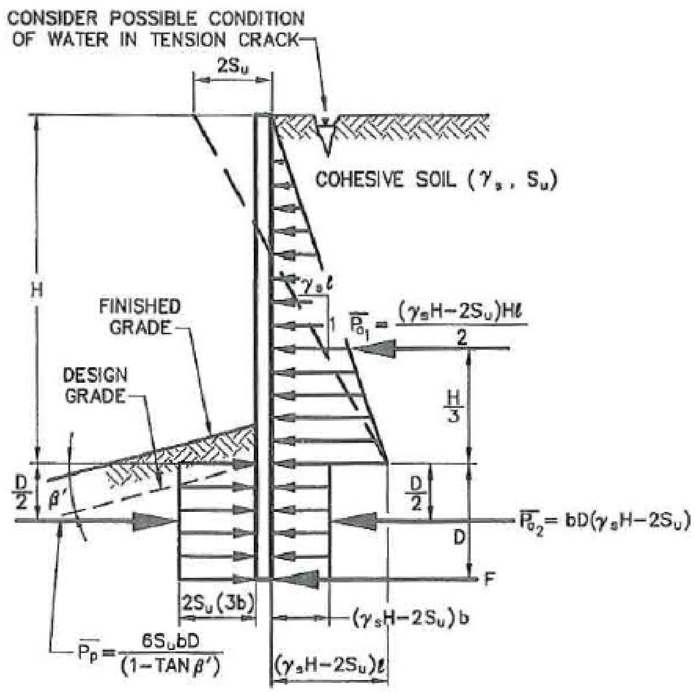
$SEC = 1$ Steel Section Modulus check, "1" indicates OK, "0" indicates No

$ComC = 1$ Compressibility of concrete check, "1" indicates OK, "0" indicates No

$D_{Check} = 1$ Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG

Objective: To evaluate the stability for Soldier Pile wall with temporary wood lagging and permanent concrete facing design.
Method: In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014 and FHWA-IF-99-015
Assumptions:

- **Soldier Pile wall is treated as nongravity cantilevered wall with discrete vertical wall elements.**
- **The height of the wall to design grade H=h+1.5 b, h is height of wall and b is shaft diameter or pile width.**
- **Load and Resistance Factors Soldier Pile wall analysis are in accordance with LRFD Bridge Design Specifications, [Sect. 3.4.1-1/3.4.1-2 and 11.5.6-1].**
- **Diagram of lateral earth pressure for nongravity cantilevered wall is assumed in accordance with LRFD Bridge Design Specifications[Figure 3.11.5.6-5].**



Load and Resistance factors:

$\gamma_{EH} := 1.5$	Load factor for horizontal earth loads (LRFD [Sect. 3.4.1-1/3.4.1-2])
$\gamma_{LS} := 1.75$	Load factor for live load surcharge (LRFD [Sect. 3.4.1-1/3.4.1-2])
$\varphi_P := 0.75$	Resistance factor for passive resistance (LRFD [Sect. 11.5.7])
$\varphi_S := 1$	Resistance factor for steel flexure (LRFD [Sect. 6.5.4.2])
$\varphi_C := 0.7$	Axial Resistance factor for compressive concrete (LRFD [Sect.5.5.4.2])
$\varphi_V := 0.9$	Resistance factor for shear and torsion (LRFD [Sect. 5.5.4.2])

Givens:

$LS := 240\ psf$		Live load surcharge

Wall Geometry:

$L := 8\ ft$		Pile spacing
$h := 9.3\ ft$		Height of wall
$b := 2.5\ ft$		Shaft diameter or pile width
$H := h + 1.5\ b$	$H = 13.1\ ft$	Design height
$D := 16\ ft$		Embedment below design grade
$\beta' := 0\ deg$		Front Slope angle at the finished grade

Soil Properties (Average):

Retained Soil:

$\varphi_1 := 0\ deg$ — Angle of internal friction

$Su_1 := 1250 \cdot psf$ — Undrained shear strength

$\gamma_1 := 110\ pcf$ — Unit weight

$Ka_1 := \frac{(1 - \sin(\varphi_1))}{(1 + \sin(\varphi_1))}$ $\quad Ka_1 = 1$ — Active earth pressure coefficient

Foundation Soil:

$\varphi_2 := 0\ deg$ — Angle of internal friction

$Su_2 := 1500\ psf$ — Undrained shear shear

$\gamma_2 := 115\ pcf$ — Unit weight

$Ka_2 := \frac{(1 - \sin(\varphi_2))}{(1 + \sin(\varphi_2))}$ $\quad Ka_2 = 1$ — Active earth pressure coefficient

Loads and Resistance:

$E_1 := \text{if}\left(\gamma_1 \cdot H - 2\ Su_1 > 0, \gamma_1 \cdot H - 2\ Su_1, 0\ psf\right)$ $\qquad E_2 := \text{if}\left(\gamma_1 \cdot H - 2\ Su_2 > 0, \gamma_1 \cdot H - 2\ Su_2, 0\ psf\right)$

$Pa_1 := \frac{E_1 \cdot H \cdot L}{2}$ $\quad Pa_1 = 0\ lbf$ — Active force-retained soil

$PaLS_1 := Ka_1 \cdot LS \cdot L \cdot H$ $\quad PaLS_1 = 25056\ lbf$ — Active force-live load surcharge on exposed height

$Pa_2 := E_2 \cdot b \cdot D$ $\quad Pa_2 = 0\ lbf$ — Active force-foundation soil

$PaLS_2 := Ka_2 \cdot LS \cdot b \cdot D$ $\quad PaLS_2 = 9600\ lbf$ — Active force-live load surcharge on embedment depth

$Pp := \frac{6\ Su_2 \cdot b \cdot D}{1 - \tan(\beta')}$ $\quad Pp = 360000\ lbf$ — Passive resistance

Moment arms from bottom:

$La_1 := D + \frac{H}{3}$ $\quad La_1 = 20.4\ ft$ — Moment arm for active force-retained soil

$LaLS_1 := D + \frac{H}{2}$ $\quad LaLS_1 = 22.5\ ft$ — Moment arm for active force-live load on exposed height

$La_2 := D - \frac{D}{2}$ $\quad La_2 = 8\ ft$ — Moment arm for active force-foundation soil

$LaLS_2 := \frac{D}{2}$ $\quad LaLS_2 = 8\ ft$ — Moment arm for active force-live load on embedment depth

$LaPp := \frac{D}{2}$ $\quad LaPp = 8\ ft$ — Moment arm for Passive resistance

Stability Check:

Disturbing moments:

$DM_1 := Pa_1 \cdot La_1$ $\quad DM_1 = 0\ lbf \cdot ft$ — Disturbing Moment of active force-retained soil

$DMLS_1 := PaLS_1 \cdot LaLS_1$ — $DMLS_1 = 564386.4\ lbf \cdot ft$ — Disturbing Moment of active force-live load acting on exposed height

$DM_2 := Pa_2 \cdot La_2$ — $DM_2 = 0\ lbf \cdot ft$ — Disturbing Moment of foundation soil

$DMLS_2 := PaLS_2 \cdot LaLS_2$ — $DMLS_2 = 76800\ lbf \cdot ft$ — Disturbing Moment of active force-live load acting on embedment depth

$M := DM_1 + DM_2$ — $M = 0\ lbf \cdot ft$ — Total disturbing moments (without live load)

$M\gamma := M \cdot \gamma_{EH} + (DMLS_1 + DMLS_1) \cdot \gamma_{LS}$ — $M\gamma = 1975352.4\ lbf \cdot ft$ — Factored total disturbing moments

Restorative moments:

$RM := Pp \cdot LaPp$ — $RM = 2880000\ lbf \cdot ft$ — Restorative moment

$RM\varphi := RM \cdot \varphi_P$ — $RM\varphi = 2160000\ lbf \cdot ft$ — Factored Restorative moment

Stability Check:

$STA := \text{if}(RM\varphi > M\gamma, 1, 0)$ — $STA = 1$ — Stability check, "1" indicates OK, "0" indicates No

Steel Section Modulus Check:

Determine the depth Z at which the shear in the wall is zero (i.e., The point at which the areas of the driving and resisting pressure diagrams are equivalent.) The maximum bending moment is at the point of zero shear.

To find the point of zero shear (Z):

$Pa_2@Z = E_2 \cdot b \cdot Z$ $\qquad La_2@Z = \frac{Z}{2}$ $\qquad La_1@Z = Z + \frac{H}{3}$

$PaLS_2@Z = Ka_2 \cdot LS \cdot b \cdot Z$ $\qquad LaLS_2@Z = \frac{Z}{2}$ $\qquad LaLS_1@Z = Z + \frac{H}{2}$

$Pp@Z = \frac{6\ Su_2 \cdot b \cdot Z}{1 - \tan(\beta')}$ $\qquad LaPp@Z = \frac{Z}{2}$

$Pa_1 + PaLS_1 + Pa_2@Z + PaLS_2@Z - Pp@Z = 0$ — Depth of zero shear below Design Grade

$$Pa_1 + PaLS_1 + E_2 \cdot b \cdot Z + Ka_2 \cdot LS \cdot b \cdot Z - \frac{6\ Su_2 \cdot b \cdot Z}{1 - \tan(\beta')} = 0$$

$$\left(E_2 \cdot b + Ka_2 \cdot LS \cdot b - \frac{6\ Su_2 \cdot b}{1 - \tan(\beta')}\right) \cdot Z + Pa_1 + PaLS_1 = 0$$

$$Z := \frac{-(Pa_1 + PaLS_1)}{\left(E_2 \cdot b + Ka_2 \cdot LS \cdot b - \frac{6\ Su_2 \cdot b}{1 - \tan(\beta')}\right)} = 1.1\ ft$$

Depth of zero shear below Design Grade

To find the maximum moment which occurs at the point of zero shear Z:

$$MaxM = \gamma_{EH} \cdot (Pa_1 \cdot La_1@Z + Pa_2@Z \cdot La_2@Z) + \gamma_{LS} \cdot (PaLS_1 \cdot LaLS_1@Z + PaLS_2@Z \cdot LaLS_2@Z) - \varphi_P \cdot Pp@Z \cdot LaPp@Z$$

$$DM@Z := \gamma_{EH} \cdot \left(Pa_1 \cdot \left(Z + \frac{H}{3}\right) + E_2 \cdot b \cdot Z \cdot \frac{Z}{2}\right) + \gamma_{LS} \cdot \left(PaLS_1 \cdot \left(Z + \frac{H}{2}\right) + (Ka_2 \cdot LS \cdot b \cdot Z) \cdot \frac{Z}{2}\right)$$

$$RM@Z := \varphi_P \cdot \left(\frac{6\ Su_2 \cdot b \cdot Z}{1 - \tan(\beta')}\right) \cdot \frac{Z}{2}$$

$MaxM := DM@Z - RM@Z$ — $MaxM = 325917.8\ lbf \cdot ft$ — Maximum moment in steel

$fy := 50\ ksi$ — Steel yield strength 50 ksi

$Sxr := \frac{MaxM}{\varphi_S\, fy}$ $Sxr = 78.2\ in^3$ — Request min. section modulus for steel

$Sx := 107\ in^3$ — Section Modulus

$SEC := \text{if}\left(Sx > Sxr, 1, 0\right)$ $SEC = 1$ — Steel Section Modulus check, "1" indicates OK, "0" indicates No

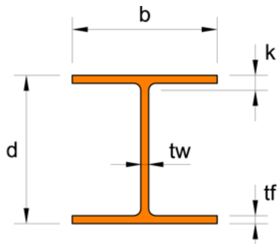
Design of timber lagging:

$S_{CTC} := L = 8\ ft$ — Center to center spacing of soldier beams

$H = 13.1\ ft$ — Design wall height

$t_{Lagging} := 100\ mm$ $t_{Lagging} = 3.9\ in$ — Timber lagging thickness is recommened based on Table 12 in FHWA-IF-99-015

Design of permanent facing:



$fc := 4000\ psi$ — Concrete compressive strength

$b := 14.6\ in$ — Flange width of HP pile

$d := 13.6\ in$ — Depth of HP pile

$k := 1.19\ in$ — Width of beam to support the lagging

$t := \frac{b}{2} - k = 6.1\ in$

$D_{Check} := \text{if}\left(D > \sqrt{b^2 + d^2} + 6\ in, 1, 0\right)$ $D_{Check} = 1$ — Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG

$\sigma_a := Ka_1 \cdot \gamma_1 \cdot H$ $\sigma_a = 10\ psi$ — Active pressure at bottom of facting- retained soil

$\sigma := \sigma_a \cdot \gamma_{EH} + Ka_1 \cdot LS \cdot \gamma_{LS}$ $\sigma = 17.9\ psi$ — Factored lateral pressure at bottome of facing Hf

$Ps := 1\ ft$ — 1' high stip pannel

$V := \sigma \cdot L \cdot Ps$ $V = 20586\ lbf$ — Factored active force

$Pr := \varphi_C \cdot fc \cdot Ps \cdot t$ $Pr = 205296\ lbf$ — Resistance force on the facing

$MmaxF := \frac{1}{10} \cdot \sigma \cdot L^2 \cdot Ps$ $MmaxF = 16468.8\ lbf \cdot ft$ — Factored max. moment at bottom of facing

$Sxc := \frac{MmaxF}{\varphi_V \cdot fc}$ $Sxc = 54.9\ in^3$ — Request section modulus of concrete

$tm := \sqrt[2]{Sxc \cdot \frac{6}{Ps}}$ $tm = 5.2\ in$ — Required thickness of facing based on moment

$ComC := \text{if}\left(Pr > V \wedge t > tm, 1, 0\right)$ $ComC = 1$ — Compressibility of concrete check, "1" indicates OK, "0" indicates No

Design results:

$STA = 1$ — Stability check, "1" indicates OK, "0" indicates No

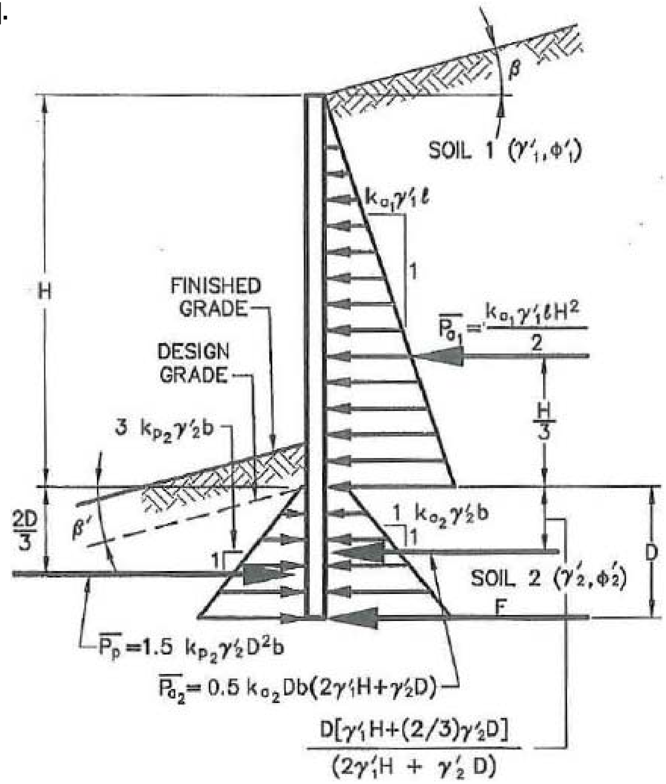
$SEC = 1$ — Steel Section Modulus check, "1" indicates OK, "0" indicates No

$ComC = 1$ — Compressibility of concrete check, "1" indicates OK, "0" indicates No

$D_{Check} = 1$ — Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG

Objective: **To evaluate the stability for Soldier Pile wall with temporary wood lagging and permanent concrete facing design.**
Method: **In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014 and FHWA-IF-99-015**
Assumptions:

- **Soldier Pile wall is treated as nongravity cantilevered wall with discrete vertical wall elements.**
- **The height of the wall to design grade H=h+1.5 b, h is height of wall and b is shaft diameter or pile width.**
- **Load and Resistance Factors Soldier Pile wall analysis are in accordance with LRFD Bridge Design Specifications, [Sect. 3.4.1-1/3.4.1-2 and 11.5.6-1].**
- **Diagram of lateral earth pressure for nongravity cantilevered wall is assumed in accordance with LRFD Bridge Design Specifications[Figure 3.11.5.6-1].**



Load and Resistance factors:

$\gamma_{EH} := 1.5$ — Load factor for horizontal earth loads (LRFD [Sect. 3.4.1-1/3.4.1-2])

$\gamma_{LS} := 1.75$ — Load factor for live load surcharge (LRFD [Sect. 3.4.1-1/3.4.1-2])

$\varphi_P := 0.75$ — Resistance factor for passive resistance (LRFD [Sect. 11.5.7])

$\varphi_S := 1$ — Resistance factor for steel flexure (LRFD [Sect. 6.5.4.2])

$\varphi_C := 0.7$ — Axial Resistance factor for compressive concrete (LRFD [Sect. 5.5.4.2])

$\varphi_V := 0.9$ — Resistance factor for shear and torsion (LRFD [Sect. 5.5.4.2])

Givens:

$LS := 240\ psf$ — Live load surcharge

Wall Geometry:

$L := 8\ ft$ — Pile spacing

$h := 9.3\ ft$ — Height of wall

$b := 2.5\ ft$ — Shaft diameter or pile width

$H := h + 1.5\ b$ $H = 13.1\ ft$ — Design height

$D := 16\ ft$ — Embedment below design grade

Soil Properties:

Retained Soil:

$\varphi_1 := 34\ deg$ — Angle of internal friction

$c_1 := 0\ psf$ — Cohesion

$\gamma_1 := 110\ pcf$ — Unit weight

$Ka_1 := \frac{(1-\sin(\varphi_1))}{(1+\sin(\varphi_1))}$ — $Ka_1 = 0.3$ — Active earth pressure coefficient

Foundation Soil:

$\varphi_2 := 34\ deg$ — Angle of internal friction

$c_2 := 0\ psf$ — Cohesion

$\gamma_2 := 115\ pcf$ — Unit weight

$Ka_2 := \frac{(1-\sin(\varphi_2))}{(1+\sin(\varphi_2))}$ — $Ka_2 = 0.3$ — Active earth pressure coefficient

$Kp_2 := \frac{1}{Ka_2}$ — $Kp_2 = 3.5$ — Passive earth pressure coefficient

Loads and Resistance:

$Pa_1 := Ka_1 \cdot \gamma_1 \cdot L \cdot \frac{H^2}{2}$ — $Pa_1 = 21184.7\ lbf$ — Active force-retained soil

$PaLS_1 := Ka_1 \cdot LS \cdot L \cdot H$ — $PaLS_1 = 7083.7\ lbf$ — Active force-live load surcharge on exposed height

$Pa_2 := \frac{1}{2}\ Ka_2 \cdot D \cdot b \cdot (2\ \gamma_1 \cdot H + \gamma_2 \cdot D)$ — $Pa_2 = 26637.4\ lbf$ — Active force-foundation soil

$PaLS_2 := Ka_2 \cdot LS \cdot b \cdot D$ — $PaLS_2 = 2714.1\ lbf$ — Active force-live load surcharge on embedment depth

$Pp := \frac{3}{2}\ Kp_2 \cdot \gamma_2 \cdot D^2 \cdot b$ — $Pp = 390499.4\ lbf$ — Passive resistance

Moment arms from bottom:

$La_1 := D + \frac{H}{3}$ — $La_1 = 20.4\ ft$ — Moment arm for active force-retained soil

$LaLS_1 := D + \frac{H}{2}$ — $LaLS_1 = 22.5\ ft$ — Moment arm for active force-live load on exposed height

$La_2 := D - D\ \frac{\gamma_1 \cdot H + \frac{2}{3}\ \gamma_2 \cdot D}{2\ \gamma_1 \cdot H + \gamma_2 \cdot D}$ — $La_2 = 7\ ft$ — Moment arm for active force-foundation soil

$LaLS_2 := \frac{D}{2}$ — $LaLS_2 = 8\ ft$ — Moment arm for active force-live load on embedment depth

$LaPp := \frac{D}{3}$ — $LaPp = 5.3\ ft$ — Moment arm for Passive resistance

Stability Check:

Disturbing moments:

$DM_1 := Pa_1 \cdot La_1$	$DM_1 = 431108.8\ lbf \cdot ft$	Disturbing Moment of active force-retained soil
$DMLS_1 := PaLS_1 \cdot LaLS_1$	$DMLS_1 = 159560.5\ lbf \cdot ft$	Disturbing Moment of active force-live load acting on exposed height
$DM_2 := Pa_2 \cdot La_2$	$DM_2 = 185355.4\ lbf \cdot ft$	Disturbing Moment of foundation soil
$DMLS_2 := PaLS_2 \cdot LaLS_2$	$DMLS_2 = 21712.5\ lbf \cdot ft$	Disturbing Moment of active force-live load acting on embedment depth
$M := DM_1 + DM_2$	$M = 616464.2\ lbf \cdot ft$	Total disturbing moments (without live load)
$M\gamma := M \cdot \gamma_{EH} + (DMLS_1 + DMLS_1) \cdot \gamma_{LS}$	$M\gamma = 1483157.9\ lbf \cdot ft$	Factored total disturbing moments

Restorative moments:

$RM := Pp \cdot LaPp$	$RM = 2082663.3\ lbf \cdot ft$	Restorative moment
$RM\varphi := RM \cdot \varphi_P$	$RM\varphi = 1561997.5\ lbf \cdot ft$	Factored Restorative moment

Stability Check:

$STA := \text{if}(RM\varphi > M\gamma, 1, 0)$	$STA = 1$	Stability check, "1" indicates OK, "0" indicates NG

Steel Section Modulus Check:

Determine the depth Z at which the shear in the wall is zero (i.e., The point at which the areas of the driving and resisting pressure diagrams are equivalent.) The maximum bending moment is at the point of zero shear.

To find the point of zero shear (Z):

$$Pa_2@Z = \frac{1}{2} Ka_2 \cdot Z \cdot b \cdot (2\ \gamma_1 \cdot H + \gamma_2 \cdot Z)$$

$$La_2@Z = Z - Z \cdot \frac{\gamma_1 \cdot H + \frac{2}{3}\gamma_2 \cdot Z}{2\ \gamma_1 \cdot H + \gamma_2 \cdot Z}$$

$$La_1@Z = Z + \frac{H}{3}$$

$$PaLS_2@Z = Ka_2 \cdot LS \cdot b \cdot Z$$

$$LaLS_2@Z = \frac{Z}{2}$$

$$LaLS_1@Z = Z + \frac{H}{2}$$

$$Pp@Z = \frac{3}{2} Kp_2 \cdot \gamma_2 \cdot Z^2 \cdot b$$

$$LaPp@Z = \frac{Z}{3}$$

$$Pa_1 + PaLS_1 + Pa_2@Z + PaLS_2@Z - Pp@Z = 0$$ Depth of zero shear below Design Grade

$$Pa_1 + PaLS_1 + \frac{1}{2} Ka_2 \cdot Z \cdot b \cdot (2\ \gamma_1 \cdot H + \gamma_2 \cdot Z) + Ka_2 \cdot LS \cdot b \cdot Z - \frac{3}{2} Kp_2 \cdot \gamma_2 \cdot Z^2 \cdot b = 0$$

$$\left(\frac{1}{2} Ka_2 \cdot b \cdot \gamma_2 - \frac{3}{2} Kp_2 \cdot \gamma_2 \cdot b\right) \cdot Z^2 + (Ka_2 \cdot b \cdot \gamma_1 \cdot H + Ka_2 \cdot LS \cdot b) \cdot Z + Pa_1 + PaLS_1 = 0$$

$$A := \left(\frac{1}{2} Ka_2 \cdot b \cdot \gamma_2 - \frac{3}{2} Kp_2 \cdot \gamma_2 \cdot b\right) \qquad B := (Ka_2 \cdot b \cdot \gamma_1 \cdot H + Ka_2 \cdot LS \cdot b) = 38101.2\ \frac{lb}{s^2} \qquad C := Pa_1 + PaLS_1$$

$$Z := \max\left(\frac{-B + \sqrt{B^2 - 4 \cdot A \cdot C}}{2 \cdot A}, \frac{-B - \sqrt{B^2 - 4 \cdot A \cdot C}}{2 \cdot A}\right) = 4.8\ ft$$ Depth of zero shear below Design Grade

To find the maximum moment which occurs at the point of zero shear Z:

$$MaxM = \gamma_{EH} \cdot \left(Pa_1 \cdot La_1@Z + Pa_2@Z \cdot La_2@Z\right) + \gamma_{LS} \cdot \left(PaLS_1 \cdot LaLS_1@Z + PaLS_2@Z \cdot LaLS_2@Z\right) - \varphi_P \cdot Pp@Z \cdot LaPp@Z$$

$$DM@Z := \gamma_{EH} \cdot \left(\left(Pa_1 \cdot \left(Z + \frac{H}{3}\right) + \left(\frac{1}{2} Ka_2 \cdot Z \cdot b \cdot \left(2\,\gamma_1 \cdot H + \gamma_2 \cdot Z\right)\right)\right) \cdot \left(Z - Z \cdot \frac{\gamma_1 \cdot H + \frac{2}{3}\gamma_2 \cdot Z}{2\,\gamma_1 \cdot H + \gamma_2 \cdot Z}\right)\right) + \gamma_{LS} \cdot \left(PaLS_1 \cdot \left(Z + \frac{H}{2}\right) + \left(Ka_2 \cdot LS \cdot b \cdot Z\right) \cdot \frac{Z}{2}\right)$$

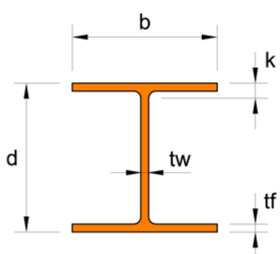
$$RM@Z := \varphi_P \cdot \left(\frac{3}{2} Kp_2 \cdot \gamma_2 \cdot Z^2 \cdot b\right) \cdot \frac{Z}{3}$$

$MaxM := DM@Z - RM@Z$	$MaxM = 411625.3\ \mathbf{lbf \cdot ft}$	Maximum moment in steel
$fy := 50\ \mathbf{ksi}$		Steel yield strength 50 ksi
$Sxr := \dfrac{MaxM}{\varphi_S\, fy}$	$Sxr = 98.8\ \mathbf{in}^3$	Request min. section modulus for steel
$Sx := 107\ \mathbf{in}^3$		Section Modulus
$SEC := \mathrm{if}\left(Sx > Sxr, 1, 0\right)$	$SEC = 1$	Steel Section Modulus check, "1" indicates OK, "0" indicates NG

Design of timber lagging:

$S_{CTC} := L = 8\ \mathbf{ft}$		Center to center spacing of soldier beams
$H = 13.1\ \mathbf{ft}$		Design wall height
$t_{Lagging} := 100\ \mathbf{mm}$	$t_{Lagging} = 3.9\ \mathbf{in}$	Timber lagging thickness is recommened based on Table 12 in FHWA-IF-99-015

Design of permanent facing:



$fc := 4000\ \mathbf{psi}$		Concrete compressive strength
$b := 14.6\ \mathbf{in}$		Flange width of HP pile
$d := 13.6\ \mathbf{in}$		Depth of HP pile
$k := 1.19\ \mathbf{in}$		
$t := \dfrac{b}{2} - k = 6.1\ \mathbf{in}$		Width of beam to support the lagging
$D_{Check} := \mathrm{if}\left(D > \sqrt{b^2 + d^2} + 6\ \mathbf{in}, 1, 0\right)$	$D_{Check} = 1$	Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG
$\sigma_a := Ka_1 \cdot \gamma_1 \cdot H$	$\sigma_a = 2.8\ \mathbf{psi}$	Active pressure at bottom of facting- retained soil
$\sigma := \sigma_a \cdot \gamma_{EH} + Ka_1 \cdot LS \cdot \gamma_{LS}$	$\sigma = 5.1\ \mathbf{psi}$	Factored lateral pressure at bottome of facing Hf
$Ps := 1\ \mathbf{ft}$		1' high stip pannel
$V := \sigma \cdot L \cdot Ps$	$V = 5820\ \mathbf{lbf}$	Factored active force
$Pr := \varphi_C \cdot fc \cdot Ps \cdot t$	$Pr = 205296\ \mathbf{lbf}$	Resistance force on the facing

$MmaxF := \frac{1}{10} \cdot \sigma \cdot L^2 \cdot Ps$ | $MmaxF = 4656\ lbf \cdot ft$ | Factored max. moment at bottom of facing

$Sxc := \frac{MmaxF}{\varphi_V \cdot fc}$ | $Sxc = 15.5\ in^3$ | Request section modulus of concrete

$tm := \sqrt[2]{Sxc \cdot \frac{6}{Ps}}$ | $tm = 2.8\ in$ | Required thickness of facing based on moment

$ComC := \text{if}\left(Pr > V \wedge t > tm, 1, 0\right)$ | $ComC = 1$ | Compressibility of concrete check, "1" indicates OK, "0" indicates No

Design results:

$STA = 1$ Stability check, "1" indicates OK, "0" indicates No

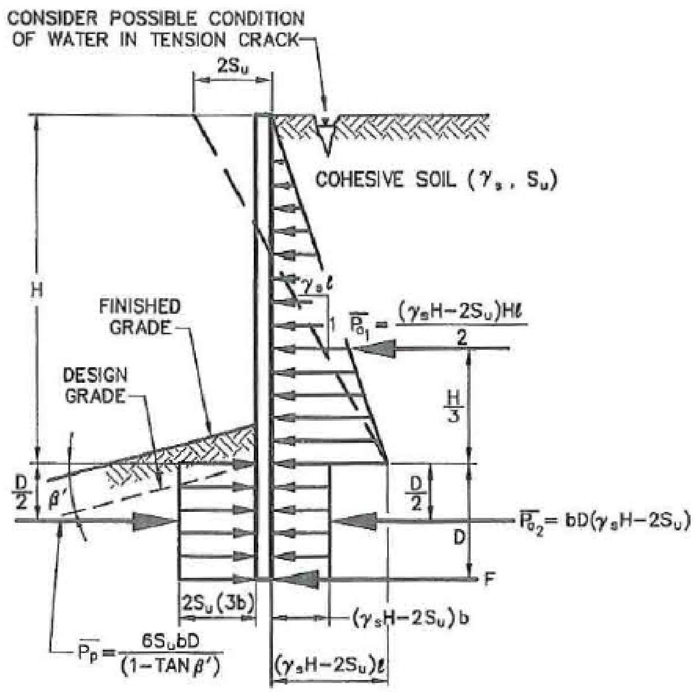
$SEC = 1$ Steel Section Modulus check, "1" indicates OK, "0" indicates No

$ComC = 1$ Compressibility of concrete check, "1" indicates OK, "0" indicates No

$D_{Check} = 1$ Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG

Objective: **To evaluate the stability for Soldier Pile wall with temporary wood lagging and permanent concrete facing design.**
Method: **In accordance with LRFD Bridge Design Specifications, 7th Ed., 2014 and FHWA-IF-99-015**
Assumptions:

- **Soldier Pile wall is treated as nongravity cantilevered wall with discrete vertical wall elements.**
- **The height of the wall to design grade H=h+1.5 b, h is height of wall and b is shaft diameter or pile width.**
- **Load and Resistance Factors Soldier Pile wall analysis are in accordance with LRFD Bridge Design Specifications, [Sect. 3.4.1-1/3.4.1-2 and 11.5.6-1].**
- **Diagram of lateral earth pressure for nongravity cantilevered wall is assumed in accordance with LRFD Bridge Design Specifications[Figure 3.11.5.6-5].**



Load and Resistance factors:

$\gamma_{EH} := 1.5$ — Load factor for horizontal earth loads (LRFD [Sect. 3.4.1-1/3.4.1-2])

$\gamma_{LS} := 1.75$ — Load factor for live load surcharge (LRFD [Sect. 3.4.1-1/3.4.1-2])

$\varphi_P := 0.75$ — Resistance factor for passive resistance (LRFD [Sect. 11.5.7])

$\varphi_S := 1$ — Resistance factor for steel flexure (LRFD [Sect. 6.5.4.2])

$\varphi_C := 0.7$ — Axial Resistance factor for compressive concrete (LRFD [Sect.5.5.4.2])

$\varphi_V := 0.9$ — Resistance factor for shear and torsion (LRFD [Sect. 5.5.4.2])

Givens:

$LS := 240\ psf$ — Live load surcharge

Wall Geometry:

$L := 8\ ft$ — Pile spacing

$h := 9.3\ ft$ — Height of wall

$b := 2.5\ ft$ — Shaft diameter or pile width

$H := h + 1.5\ b$ $\quad H = 13.1\ ft$ — Design height

$D := 16\ ft$ — Embedment below design grade

$\beta' := 0\ deg$ — Front Slope angle at the finished grade

Soil Properties (Average):

Retained Soil:

$\varphi_1 := 0\ deg$ — Angle of internal friction

$Su_1 := 2250 \cdot psf$ — Undrained shear strength

$\gamma_1 := 110\ pcf$ — Unit weight

$Ka_1 := \frac{(1-\sin(\varphi_1))}{(1+\sin(\varphi_1))}$ $\quad Ka_1 = 1$ — Active earth pressure coefficient

Foundation Soil:

$\varphi_2 := 0\ deg$ — Angle of internal friction

$Su_2 := 1550\ psf$ — Undrained shear shear

$\gamma_2 := 120\ pcf$ — Unit weight

$Ka_2 := \frac{(1-\sin(\varphi_2))}{(1+\sin(\varphi_2))}$ $\quad Ka_2 = 1$ — Active earth pressure coefficient

Loads and Resistance:

$E_1 := \text{if}\left(\gamma_1 \cdot H - 2\ Su_1 > 0, \gamma_1 \cdot H - 2\ Su_1, 0\ psf\right)$ $\quad E_2 := \text{if}\left(\gamma_1 \cdot H - 2\ Su_2 > 0, \gamma_1 \cdot H - 2\ Su_2, 0\ psf\right)$

$Pa_1 := \frac{E_1 \cdot H \cdot L}{2}$ $\quad Pa_1 = 0\ lbf$ — Active force-retained soil

$PaLS_1 := Ka_1 \cdot LS \cdot L \cdot H$ $\quad PaLS_1 = 25056\ lbf$ — Active force-live load surcharge on exposed height

$Pa_2 := E_2 \cdot b \cdot D$ $\quad Pa_2 = 0\ lbf$ — Active force-foundation soil

$PaLS_2 := Ka_2 \cdot LS \cdot b \cdot D$ $\quad PaLS_2 = 9600\ lbf$ — Active force-live load surcharge on embedment depth

$Pp := \frac{6\ Su_2 \cdot b \cdot D}{1-\tan(\beta')}$ $\quad Pp = 372000\ lbf$ — Passive resistance

Moment arms from bottom:

$La_1 := D + \frac{H}{3}$ $\quad La_1 = 20.4\ ft$ — Moment arm for active force-retained soil

$LaLS_1 := D + \frac{H}{2}$ $\quad LaLS_1 = 22.5\ ft$ — Moment arm for active force-live load on exposed height

$La_2 := D - \frac{D}{2}$ $\quad La_2 = 8\ ft$ — Moment arm for active force-foundation soil

$LaLS_2 := \frac{D}{2}$ $\quad LaLS_2 = 8\ ft$ — Moment arm for active force-live load on embedment depth

$LaPp := \frac{D}{2}$ $\quad LaPp = 8\ ft$ — Moment arm for Passive resistance

Stability Check:

Disturbing moments:

$DM_1 := Pa_1 \cdot La_1$ $\quad DM_1 = 0\ lbf \cdot ft$ — Disturbing Moment of active force-retained soil

$DMLS_1 := PaLS_1 \cdot LaLS_1$ | $DMLS_1 = 564386.4\ lbf \cdot ft$ | Disturbing Moment of active force-live load acting on exposed height

$DM_2 := Pa_2 \cdot La_2$ | $DM_2 = 0\ lbf \cdot ft$ | Disturbing Moment of foundation soil

$DMLS_2 := PaLS_2 \cdot LaLS_2$ | $DMLS_2 = 76800\ lbf \cdot ft$ | Disturbing Moment of active force-live load acting on embedment depth

$M := DM_1 + DM_2$ | $M = 0\ lbf \cdot ft$ | Total disturbing moments (without live load)

$M\gamma := M \cdot \gamma_{EH} + (DMLS_1 + DMLS_1) \cdot \gamma_{LS}$ | $M\gamma = 1975352.4\ lbf \cdot ft$ | Factored total disturbing moments

Restorative moments:

$RM := Pp \cdot LaPp$ | $RM = 2976000\ lbf \cdot ft$ | Restorative moment

$RM\varphi := RM \cdot \varphi_P$ | $RM\varphi = 2232000\ lbf \cdot ft$ | Factored Restorative moment

Stability Check:

$STA := \text{if}(RM\varphi > M\gamma, 1, 0)$ | $STA = 1$ | Stability check, "1" indicates OK, "0" indicates No

Steel Section Modulus Check:

Determine the depth Z at which the shear in the wall is zero (i.e., The point at which the areas of the driving and resisting pressure diagrams are equivalent.) The maximum bending moment is at the point of zero shear.

To find the point of zero shear (Z):

$$Pa_2@Z = E_2 \cdot b \cdot Z \qquad La_2@Z = \frac{Z}{2} \qquad La_1@Z = Z + \frac{H}{3}$$

$$PaLS_2@Z = Ka_2 \cdot LS \cdot b \cdot Z \qquad LaLS_2@Z = \frac{Z}{2} \qquad LaLS_1@Z = Z + \frac{H}{2}$$

$$Pp@Z = \frac{6\ Su_2 \cdot b \cdot Z}{1 - \tan(\beta')} \qquad LaPp@Z = \frac{Z}{2}$$

$$Pa_1 + PaLS_1 + Pa_2@Z + PaLS_2@Z - Pp@Z = 0$$

Depth of zero shear below Design Grade

$$Pa_1 + PaLS_1 + E_2 \cdot b \cdot Z + Ka_2 \cdot LS \cdot b \cdot Z - \frac{6\ Su_2 \cdot b \cdot Z}{1 - \tan(\beta')} = 0$$

$$\left(E_2 \cdot b + Ka_2 \cdot LS \cdot b - \frac{6\ Su_2 \cdot b}{1 - \tan(\beta')}\right) \cdot Z + Pa_1 + PaLS_1 = 0$$

$$Z := \frac{-(Pa_1 + PaLS_1)}{\left(E_2 \cdot b + Ka_2 \cdot LS \cdot b - \frac{6\ Su_2 \cdot b}{1 - \tan(\beta')}\right)} = 1.1\ ft$$

Depth of zero shear below Design Grade

To find the maximum moment which occurs at the point of zero shear Z:

$$MaxM = \gamma_{EH} \cdot (Pa_1 \cdot La_1@Z + Pa_2@Z \cdot La_2@Z) + \gamma_{LS} \cdot (PaLS_1 \cdot LaLS_1@Z + PaLS_2@Z \cdot LaLS_2@Z) - \varphi_P \cdot Pp@Z \cdot LaPp@Z$$

$$DM@Z := \gamma_{EH} \cdot \left(Pa_1 \cdot \left(Z + \frac{H}{3}\right) + E_2 \cdot b \cdot Z \cdot \frac{Z}{2}\right) + \gamma_{LS} \cdot \left(PaLS_1 \cdot \left(Z + \frac{H}{2}\right) + (Ka_2 \cdot LS \cdot b \cdot Z) \cdot \frac{Z}{2}\right)$$

$$RM@Z := \varphi_P \cdot \left(\frac{6\ Su_2 \cdot b \cdot Z}{1 - \tan(\beta')}\right) \cdot \frac{Z}{2}$$

$MaxM := DM@Z - RM@Z$ | $MaxM = 324587\ lbf \cdot ft$ | Maximum moment in steel

$fy := 50\ \mathbf{ksi}$ — Steel yield strength 50 ksi

$Sxr := \dfrac{MaxM}{\varphi_S\, fy}$ — $Sxr = 77.9\ \mathbf{in}^3$ — Request min. section modulus for steel

$Sx := 107\ \mathbf{in}^3$ — Section Modulus

$SEC := \text{if}\left(Sx > Sxr, 1, 0\right)$ — $SEC = 1$ — Steel Section Modulus check, "1" indicates OK, "0" indicates No

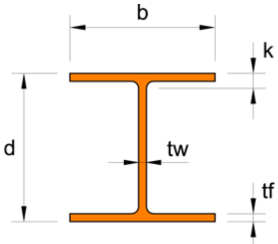
Design of timber lagging:

$S_{CTC} := L = 8\ \mathbf{ft}$ — Center to center spacing of soldier beams

$H = 13.1\ \mathbf{ft}$ — Design wall height

$t_{Lagging} := 100\ \mathbf{mm}$ — $t_{Lagging} = 3.9\ \mathbf{in}$ — Timber lagging thickness is recommened based on Table 12 in FHWA-IF-99-015

Design of permanent facing:



$fc := 4000\ \mathbf{psi}$ — Concrete compressive strength

$b := 14.6\ \mathbf{in}$ — Flange width of HP pile

$d := 13.6\ \mathbf{in}$ — Depth of HP pile

$k := 1.19\ \mathbf{in}$ — Width of beam to support the lagging

$t := \dfrac{b}{2} - k = 6.1\ \mathbf{in}$

$D_{Check} := \text{if}\left(D > \sqrt{b^2 + d^2} + 6\ \mathbf{in}, 1, 0\right)$ — $D_{Check} = 1$ — Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG

$\sigma_a := Ka_1 \cdot \gamma_1 \cdot H$ — $\sigma_a = 10\ \mathbf{psi}$ — Active pressure at bottom of facting- retained soil

$\sigma := \sigma_a \cdot \gamma_{EH} + Ka_1 \cdot LS \cdot \gamma_{LS}$ — $\sigma = 17.9\ \mathbf{psi}$ — Factored lateral pressure at bottome of facing Hf

$Ps := 1\ \mathbf{ft}$ — 1' high stip pannel

$V := \sigma \cdot L \cdot Ps$ — $V = 20586\ \mathbf{lbf}$ — Factored active force

$Pr := \varphi_C \cdot fc \cdot Ps \cdot t$ — $Pr = 205296\ \mathbf{lbf}$ — Resistance force on the facing

$MmaxF := \dfrac{1}{10} \cdot \sigma \cdot L^2 \cdot Ps$ — $MmaxF = 16468.8\ \mathbf{lbf \cdot ft}$ — Factored max. moment at bottom of facing

$Sxc := \dfrac{MmaxF}{\varphi_V \cdot fc}$ — $Sxc = 54.9\ \mathbf{in}^3$ — Request section modulus of concrete

$tm := \sqrt[2]{Sxc \cdot \dfrac{6}{Ps}}$ — $tm = 5.2\ \mathbf{in}$ — Required thickness of facing based on moment

$ComC := \text{if}\left(Pr > V \wedge t > tm, 1, 0\right)$ — $ComC = 1$ — Compressibility of concrete check, "1" indicates OK, "0" indicates No

Design results:

$STA = 1$ — Stability check, "1" indicates OK, "0" indicates No

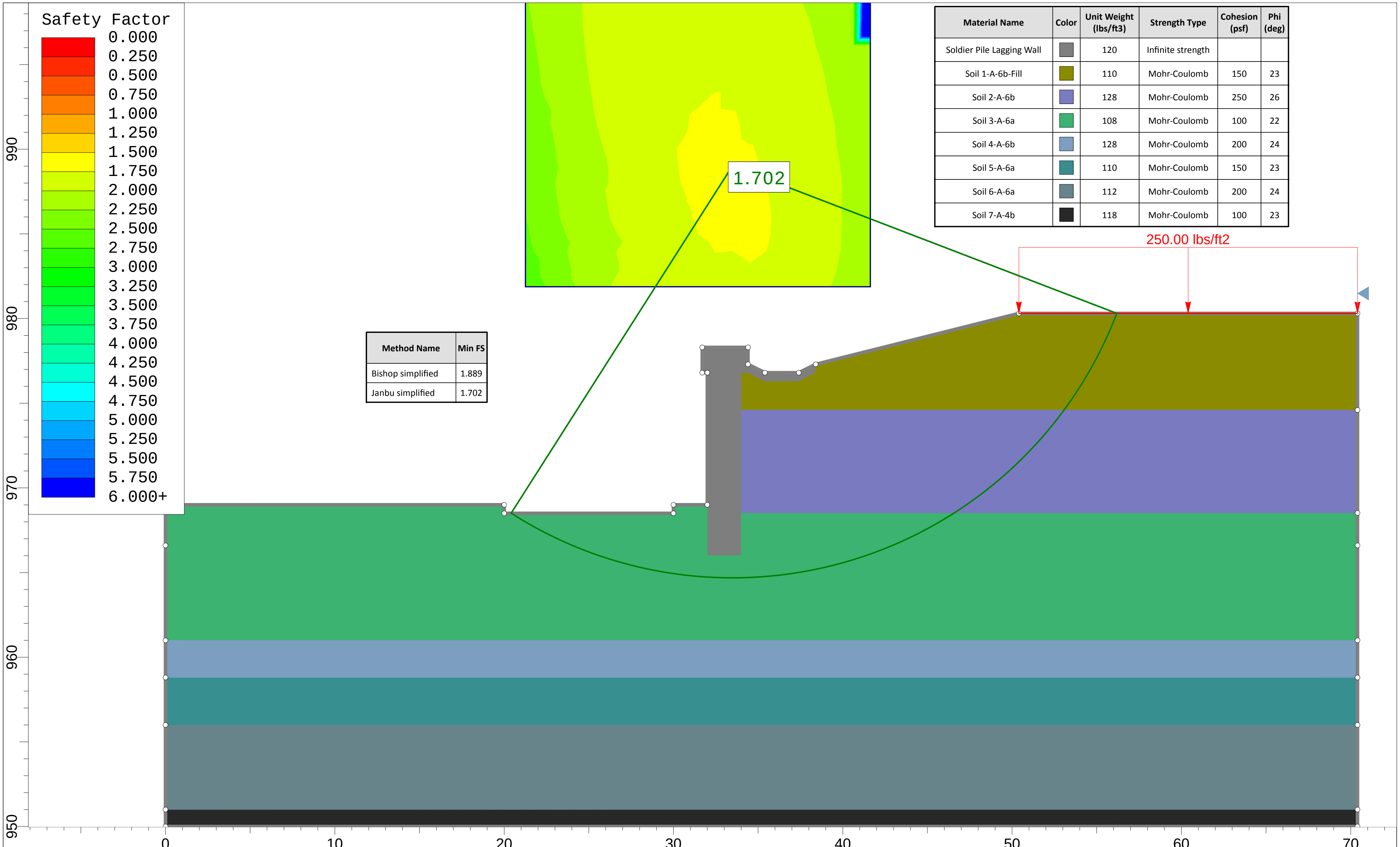
$SEC = 1$ — Steel Section Modulus check, "1" indicates OK, "0" indicates No

$ComC = 1$ — Compressibility of concrete check, "1" indicates OK, "0" indicates No

$D_{Check} = 1$ — Shaft diameter check, 3 in concrete cover in any direction, "1" indicates OK, "0" indicates NG

RETAINING WALL #4

GLOBAL STABILITY ANALYSIS

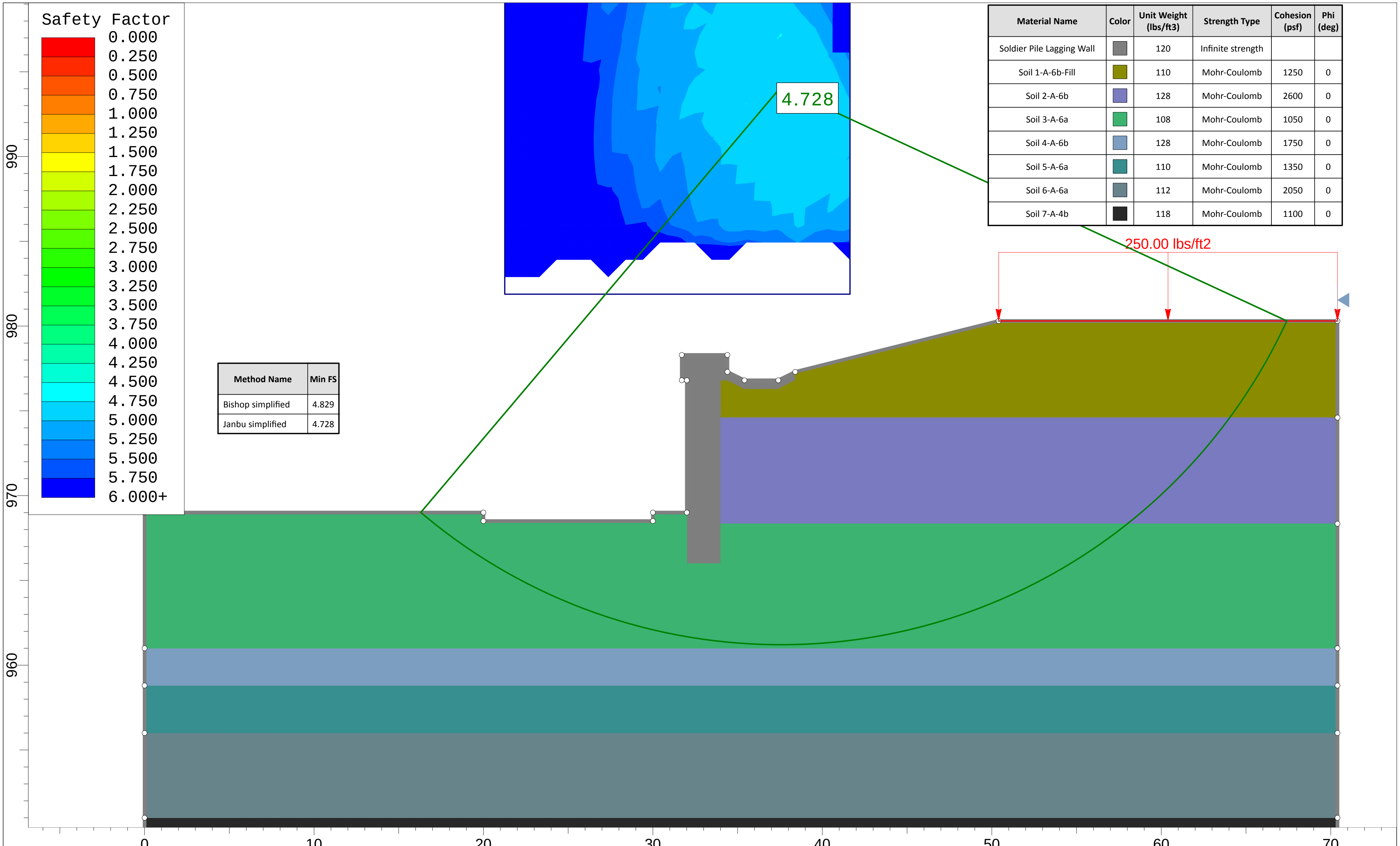


Method Name	Min FS
Bishop simplified	1.889
Janbu simplified	1.702

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Soldier Pile Lagging Wall		120	Infinite strength		
Soil 1-A-6b-Fill		110	Mohr-Coulomb	150	23
Soil 2-A-6b		128	Mohr-Coulomb	250	26
Soil 3-A-6a		108	Mohr-Coulomb	100	22
Soil 4-A-6b		128	Mohr-Coulomb	200	24
Soil 5-A-6a		110	Mohr-Coulomb	150	23
Soil 6-A-6a		112	Mohr-Coulomb	200	24
Soil 7-A-4b		118	Mohr-Coulomb	100	23

Project	MED-18 RW 4 at Goodwill STA 168+50
Analysis Description	Global Stability Analysis-Circular-Effective stress
Drawn By	ZM
Scale	1:94
Company	National Engineering & Architectural Services, Inc
Date	12/30/2016, 10:59:20 AM
File Name	RW4_STA168+50_Circular_Effective.slim

SLIDEINTERPRET 7.021



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Soldier Pile Lagging Wall		120	Infinite strength		
Soil 1-A-6b-Fill		110	Mohr-Coulomb	1250	0
Soil 2-A-6b		128	Mohr-Coulomb	2600	0
Soil 3-A-6a		108	Mohr-Coulomb	1050	0
Soil 4-A-6b		128	Mohr-Coulomb	1750	0
Soil 5-A-6a		110	Mohr-Coulomb	1350	0
Soil 6-A-6a		112	Mohr-Coulomb	2050	0
Soil 7-A-4b		118	Mohr-Coulomb	1100	0

Method Name	Min FS
Bishop simplified	4.829
Janbu simplified	4.728

Project	MED-18 RW 4 at Goodwill STA 168+50		
Analysis Description	Global Stability Analysis-Circular-Total stress		
Drawn By ZM	*Scale* 1:94	*Company*	National Engineering & Architectural Services, Inc
Date 12/30/2016, 10:59:20 AM		*File Name*	RW4_STA168+50_Circular_Total.slim

SLIDEINTERPRET 7.021

APPENDIX E

SEISMIC ANALYSIS

USGS Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (41.13641°N, 81.81163°W)

Site Class D – "Stiff Soil"

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B. Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 [1]	PGA = 0.042 g
From Figure 3.4.1-3 [2]	S_S = 0.090 g
From Figure 3.4.1-4 [3]	S_1 = 0.032 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1–1 Site Class Definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \le 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \le 2{,}500$	$\bar{N} > 50$	>2,000 psf
D	Stiff soil profile	$600 \le \bar{v}_s < 1{,}200$	$15 \le \bar{N} \le 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	<1,000 psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \ge 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	$\boxed{1.6}$	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.042 g, F_{PGA} = 1.600

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	$\boxed{1.6}$	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.090 g, F_a = 1.600

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_1

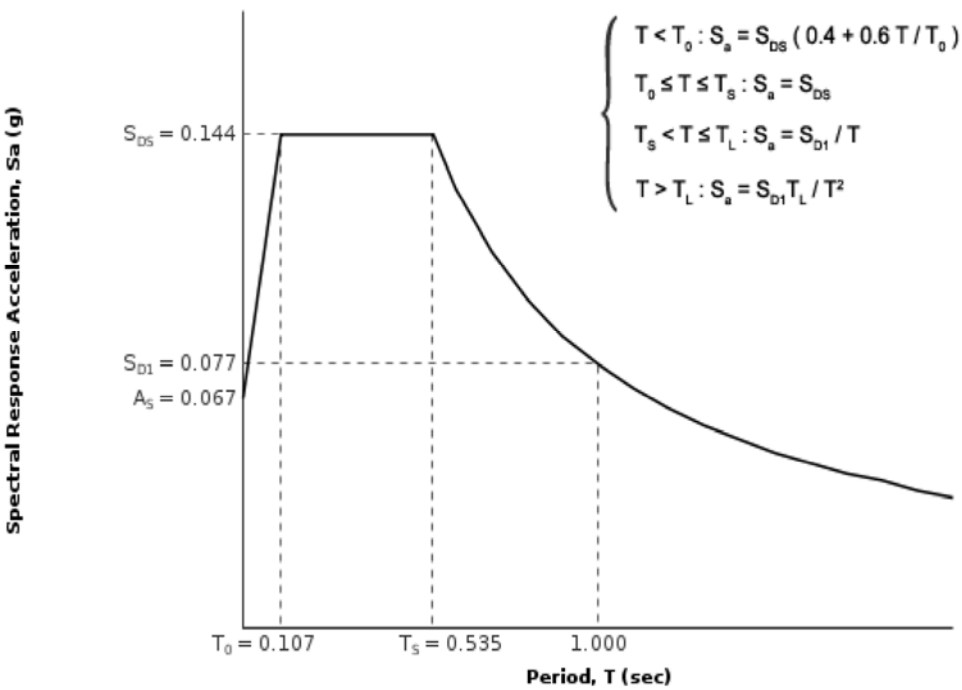
For Site Class = D and S_1 = 0.032 g, F_v = 2.400

Equation (3.4.1-1): $A_S = F_{PGA}\ PGA = 1.600 \times 0.042 = 0.067$ g

Equation (3.4.1-2): $S_{DS} = F_a\ S_S = 1.600 \times 0.090 = 0.144$ g

Equation (3.4.1-3): $S_{D1} = F_v\ S_1 = 2.400 \times 0.032 = 0.077$ g

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	**SDC**
$S_{D1} < 0.15g$	A
$0.15g \le S_{D1} < 0.30g$	B
$0.30g \le S_{D1} < 0.50g$	C
$0.50g \le S_{D1}$	D

For S_{D1} = 0.077 g, Seismic Design Category = A

Seismic Design Category ≡ “the design category in accordance with Table 3.5-1” = A

References

1. *Figure 3.4.1-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf
2. *Figure 3.4.1-3*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf
3. *Figure 3.4.1-4*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf